(2)

TECHNICAL REPORT GL-91-3

US Army Corps of Engineers

THE EFFECTS OF NATURAL SANDS ON ASPHALT CONCRETE ENGINEERING PROPERTIES

by

Randy C. Ahlrich

Geotechnical Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

March 1991
Final Report

GEOTECHNICAL LABORATORY

Prepared for DEPARTMENT OF THE AIR FORCE
Air Force Engineering and Services Center
Tyndall Air Force Base, Florida 32403-6001

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1. AGENCY USE ONLY *(Leave blank)*

2. REPORT DATE
March 1991

3. REPORT TYPE AND DATES COVERED
Final report

4. TITLE AND SUBTITLE
The Effects of Natural Sands on Asphalt Concrete Engineering Properties

5. FUNDING NUMBERS

6. AUTHOR(S)
Randy C. Ahlrich

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
USAE Waterways Experiment Station, Geotechnical Laboratory,
3909 Halls Ferry Road, Vicksburg, MS 39180-6199

8. PERFORMING ORGANIZATION REPORT NUMBER
Technical Report GL-91-3

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
Air Force Engineering and Services Center
Tyndall Air Force Base, FL 32403-6001

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

12a. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release; distribution unlimited

12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

Asphalt concrete rutting is premature deformation that develops in wheelpaths under channelized loads. Asphalt concrete mixtures that exhibit plastic flow are caused by asphalt concrete mixtures that have an excessive asphalt content and/or excessive amount of uncrushed rounded aggregate.

This laboratory study was conducted to evaluate the effects of natural sands on the engineering properties of asphalt concrete. This research consisted of a literature review and a two-phase laboratory study on laboratory-produced specimens. Conventional and state-of-the-art testing procedures including indirect tensile, resilient modulus, and unconfined creep rebound were used to determine the effects of natural sands.

The conclusions of the laboratory study indicated that use of natural sand materials decreased strength characteristics of asphalt concrete mixtures. Replacing natural sand materials with crushed aggregates increased the resistance to permanent deformation. This study recommends that the maximum limit for natural sand be 15 percent, but to maximize the decrease in rutting potential, all crushed aggregate should be used.

14. SUBJECT TERMS

Aggregate	Indirect tensile	Resilient modulus
Asphalt concrete	Natural sands	Rutting
Gyratory testing	Permanent deformation	Unconfined creep

15. NUMBER OF PAGES
172

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
Unclassified	Unclassified	Unclassified	

PREFACE

This study was conducted by the Geotechnical Laboratory (GL), US Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, for the Air Force Engineering and Services Center (AFESC), US Air Force. This work was conducted from October 1989 to December 1990 under the project entitled, "Effects of Natural Sand on the Rutting of Asphalt Concrete Mixtures." The AFESC Technical Monitor was Mr. Jim Greene.

The study was conducted under the general supervision of Dr. W. F. Marcuson III, Chief, GL; Mr. H. H. Ulery, Jr., Chief, Pavement Systems Division (PSD), GL; and Dr. R. S. Rollings and Mr. L. N. Godwin, former Chiefs, Materials Research and Construction Technology (MRCT) Branch, PSD. This report was produced under direct supervision of Mr. T. W. Vollor, Acting Chief, MRCT Branch, PSD. Personnel engaged in the testing, evaluating, and analysis of this project included Messrs. T. Carr, J. Duncan, R. Graham, T. McCaffrey, H. McKnight, J. Simmons, and D. Reed. The Principal Investigator was Mr. R. C. Ahlrich who also wrote this report.

COL Larry B. Fulton, EN, is Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

FIGURE PAGE

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
Fahrenheit degrees	5/9	Celsius degrees or Kelvins *
inches	2.54	centimetres
pounds (force)	4.448222	newtons
pounds (force) per square inch	6.894757	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

The Effects of Natural Sands on Asphalt

Concrete Engineering Properties

CHAPTER I

INTRODUCTION

Background

In recent years, deterioration of asphalt concrete pavements on military installations and state highways has increased. This deterioration has been caused by higher traffic volumes, higher traffic loads, increasing tire pressures, poor construction quality control and decreased quality of asphalt concrete mixtures. Rutting is one of the most common forms of deterioration in asphalt concrete pavement (6).

Asphalt concrete rutting is generally premature longitudinal deformation that develops in the wheelpaths under channelized loads. Rutting of asphalt concrete pavements is a complicated process and can be caused by several factors. Rutting is typically caused by one of the following: 1) shear deformation of base course or subgrade, 2) densification or consolidation of base course or subgrade, 3) densification or consolidation of asphalt concrete material, and 4) plastic flow of asphalt concrete material (2).

Rutting of an asphalt concrete pavement caused by plastic flow of the asphalt concrete material indicates a problem with the asphalt concrete mixture. Plastic flow of an asphalt concrete material illustrates an unstable mixture. Rutting of this nature is demonstrated by a depression under the loaded area with humps on either side. Asphalt concrete mixtures that exhibit plastic flow rutting are generally caused by asphalt concrete mixtures that have an excessive asphalt content, an improper gradation, and/or an excessive amount of uncrushed rounded aggregate.

Uncrushed rounded aggregates have been proven to decrease the strength properties of asphalt concrete mixtures and produce materials that are unstable. Natural sand materials, which are primarily uncrushed rounded particles, are often used in asphalt concrete mixtures because these materials are generally less expensive, readily available, and can be blended easily with other materials. Natural sand materials have a smooth, rounded surface texture that greatly reduces the interlocking properties of the asphalt concrete and reduces the strength properties. Low strength properties and stability values in asphalt concrete mixtures allow deformation to occur, which leads to rutting (18,20,22,24).

In numerous field evaluations by the Federal Highway Administration (FHWA) and U.S. Corps of Engineers of asphalt concrete pavements that had exhibited rutting, it was found that many highway departments and military installations were allowing an excess of natural sand in their asphalt concrete mixtures. This excessive amount of uncrushed rounded particles was causing a reduction in pavement strength and stability and an increase in permanent deformation under traffic (2,5,19,27,31).

Most agencies that construct flexible pavements have some guidance or have set allowable limits on the use of natural sands. The Corps of Engineers has set allowable limits for natural sand content for asphalt concrete mixtures, but these limits are not widely used outside major airfield paving projects (9). Some state highway departments have also set limits for the amount of natural sand, but the maximum limit varies from 10 to 30 percent. Some other highway departments have no limits and allow an unlimited amount of natural sand. The general consensus is that the maximum limit for natural sand is not generally controlled.

The natural sand limits established by the Corps of Engineers are based on past observed behavior and performance in the field. Laboratory evaluations have not been conducted to determine allowable limits for natural sands. Since the widely specified Marshall mix design procedure does not always reflect the detrimental effect of natural sand, many mix designs produced for state highway departments and military installations have an excess amount of natural sand.

Purpose

The purpose of this research was to evaluate the effects of natural sands on the engineering properties of asphalt concrete. This research provided a sound basis for selecting allowable natural sand contents for asphalt concrete mixtures to increase strength and stability and decrease the rutting potential. The documentation of this work provided strong support for the wide use of natural sand content limits in asphalt concrete mixtures. This information has the potential to improve the rutting performance of asphalt concrete at a negligible additional cost compared to other more costly approaches such as asphalt binder modifiers.

Objective

The objective of this research is to determine the influence of various amounts of natural sands on the engineering properties of asphalt concrete mixtures and to set quantitative limits of natural sand to prevent unstable mixtures and reduce rutting potential.

Scope

The scope of this research study included a review of available literature and existing data, a two-phase laboratory study on laboratory-produced samples, and an analysis of the data. Both conventional and state-of-the-art testing procedures were incorporated into the laboratory test plan. Asphalt concrete mixture tests that were performed included the Marshall stability and flow, indirect tensile, resilient modulus and unconfined creep-rebound tests. A diagram of the laboratory test plan used in this study is shown in Figure 1.

To evaluate the effect of natural sands on asphalt concrete mixtures, a laboratory study was conducted using two gradations of natural sand material with four different percentages of sand in the asphalt concrete mixtures. The asphalt concrete mixtures were produced with 0, 10, 20, and 30 percent natural sand. Each aggregate blend was fabricated in the laboratory with a constant mixture gradation.

The test plan for the natural sand laboratory evaluation is summarized in Table 1. Phase I of the laboratory study involved testing the laboratory materials (aggregates, sands, and asphalt cement) and conducting mix designs for the seven asphalt concrete mixtures using the Marshall mix design criteria. All asphalt concrete samples were compacted with the Corps of Engineers Gyratory Testing Machine (GTM) using 200 psi pressure, 1-degree gyration angle, and 30 revolutions which is equivalent to the 75-blow Marshall hand hammer compactive effort. The optimum asphalt content for each aggregate blend was selected at 4 percent voids total mix in the asphalt concrete mixtures.

Phase II of the laboratory study involved conducting a series of laboratory tests to determine the engineering properties of the seven asphalt concrete mixtures. Forty specimens at the optimum asphalt content were produced for each aggregate blend. The following laboratory tests were conducted on the specimens:

1. Marshall stability and flow at 140^0F.
2. Indirect tensile at 77^0F and 104^0F.
3. Resilient modulus at 77^0F and 104^0F.
4. Unconfined creep-rebound at 77^0F and 104^0F.

Several repetitions of each test were performed in order to provide sufficient data for a complete analysis. A total of 280 specimens were analyzed. From this series of tests, the effects of natural sands on the engineering properties were determined.

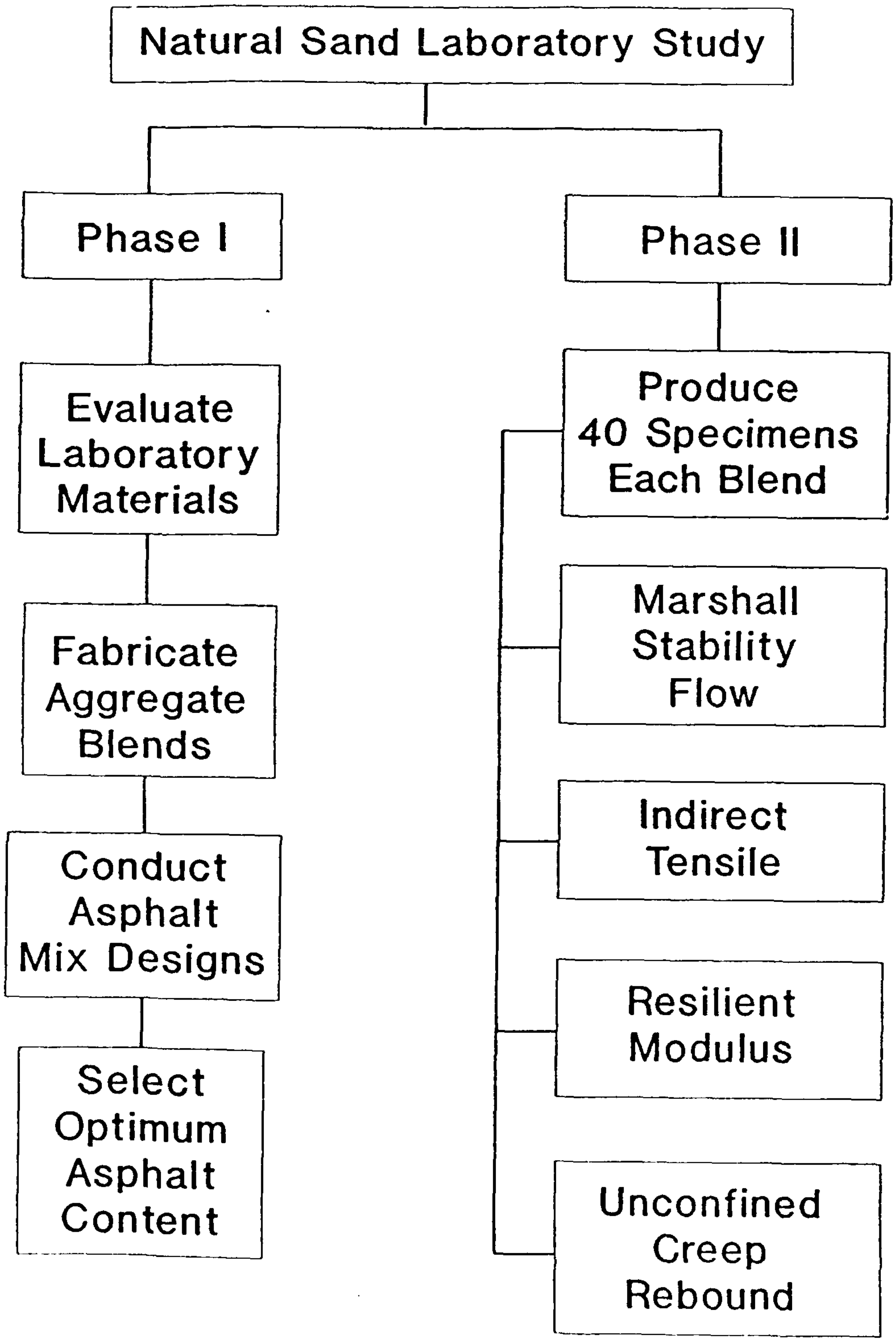


Figure 1. Flow Diagram of Natural Sand Laboratory Study

TABLE 1

NATURAL SAND LABORATORY STUDY TEST PLAN

1. Phase I - Material Evaluation and Mix Designs

   a. Test laboratory materials - aggregates, sands, asphalt cement

   b. Select natural sand materials - mason and concrete

   c. Produce aggregate blends for various percentages of sand - 0, 10, 20, 30

   d. Conduct seven asphalt concrete mix designs with laboratory limestone labstock and two natural sands

   e. Compact all asphalt concrete specimens with Gyratory Testing Machine (GTM)

   f. Select optimum asphalt content at 4 percent voids total mix

2. Phase II - Laboratory Evaluation

   a. Produce 40 specimens at optimum asphalt content for seven aggregate blends - Total of 280 specimens

   b. Designations for seven aggregate blends

Blend	Material
S-0	100 percent limestone
S-1M	90 percent limestone - 10 percent mason sand
S-2M	80 percent limestone - 20 percent mason sand
S-3M	70 percent limestone - 30 percent mason sand
S-1C	90 percent limestone - 10 percent concrete sand
S-2C	80 percent limestone - 20 percent concrete sand
S-3C	70 percent limestone - 30 percent concrete sand

   c. Conduct the following test on each aggregate blend

      1. Marshall stability and flow at 140^0F
      2. Indirect tensile at 77^0F and 104^0F
      3. Resilient modulus at 77^0F and 104^0F
      4. Unconfined creep-rebound at 77^0F and 104^0F

CHAPTER II

REVIEW OF LITERATURE

One of the most serious problems affecting our road system today is rutting of asphalt concrete pavements. For the last 15 years, state highway departments throughout the country have reported an increase in premature rutting (2). Many studies and evaluations have been conducted to determine the causes of rutting. During the development of the Marshall procedure at the Waterways Experiment Station (WES)(21), evaluations indicated that the characteristics of the fine aggregate control the capacity of dense-graded asphalt concrete mixtures to resist traffic-induced stresses that cause rutting.

Brown (6) indicated several factors contributed to the potential problems that produce rutting. The factors listed included excessive asphalt content, excessive use of natural sand, improperly crushed aggregate, and low field density. Laboratory studies and field evaluations conducted in the states of Wyoming (31), New Mexico (19), and Florida (27) also identified excessive sand-size particles and rounded aggregates as two factors that caused rutting in asphalt concrete pavements.

Numerous laboratory research studies have been conducted comparing crushed coarse and fine aggregates to natural or uncrushed aggregates in asphalt concrete mixtures. Many of the laboratory evaluations were performed during the 1950's and 1960's. Herrin and Goetz (20) evaluated the effect of aggregate shape on the stability of asphalt concrete materials. This research involved crushed and uncrushed gravel, crushed limestone for the coarse aggregate, and natural sand and crushed limestone sand for the fine aggregate. The primary conclusion was that the strength of the mixture, regardless of the type of coarse aggregate, increased substantially when fine aggregate was changed from rounded natural sand to crushed limestone. A secondary conclusion was that the strength of the mixture was affected more by a change in the fine aggregate than a change in the coarse aggregate.

In 1961, Wedding and Gaynor (30) researched the effect of aggregate particle shape in well-graded asphalt concrete mixtures. The percentages of crushed coarse aggregates and the types of fine aggregates which included natural and washed concrete sands were varied in the mixtures. Comparisons of these different aggregate blends were conducted on specimens produced using the Marshall procedure. Mixtures with crushed aggregates produced higher stability values than mixtures with uncrushed, rounded aggregates. The substitution of all crushed aggregate for natural sand and gravel also increased the stability approximately 45 percent.

Griffith and Kallas (17,18) researched the effects of aggregate types on void and strength characteristics of asphalt concrete mixtures. Uncrushed gravel mixtures were found to develop voids lower than the voids in crushed aggregates mixtures. Griffith and Kallas also evaluated the influence of fine aggregates on the strength of asphalt concrete specimens. Combinations of aggregate blends with natural and crushed coarse aggregate and natural sand fine aggregate were analyzed. An increase in angularity or crushed faces increased the Hveem and Marshall stability values at optimum asphalt content. An increase in angularity in the fine aggregates also increased the minimum void percentages and increased optimum asphalt contents.

Shklarsky and Livneh (29) conducted a study evaluating the difference between uncrushed and crushed coarse aggregate combined with natural sand and crushed fine aggregate. Replacing natural sand materials with crushed fine aggregate increased the stability and strength properties in Marshall specimens and reduced permanent deformation, improved resistance to water, reduced asphalt cement sensitivity, and increased voids. Shklarsky and Livneh also concluded that replacing uncrushed coarse aggregate with crushed material did not significantly improve the asphalt concrete mixture.

Kalcheff and Tunnicliff (22) researched the effects of coarse aggregate gradations, shape effects of fine aggregates, and effects of high mineral filler content. Asphalt concrete specimens were produced using the Marshall and Hveem procedures with aggregate blends composed of natural and manufactured (crushed) sands. The optimum asphalt content was approximately the same for natural sand mixtures and manufactured sand mixtures if the sands had similar particle shape. The optimum asphalt content would be higher if the manufactured sand had more angular particles. Also, mixtures containing crushed coarse and fine aggregates were more resistant to permanent deformation from repeated traffic loadings, and much less susceptible to the effects of temperature than comparable mixtures containing natural sand.

Button and Perdomo (8) conducted a laboratory study that was designed to evaluate the effects of natural sands on permanent deformation and o quantify the influence on resistance to plastic deformation when natural sand is replaced with crushed aggregate. Increases in total deformation occurred as the percentage of natural sand increased. The texture, shape, and porosity of the fine aggregate were major factors controlling plastic deformation in asphalt concrete mixtures. They recommended replacing the natural sand material with manufactured sand to increase the resistance of the asphalt concrete pavement to permanent deformation.

Marks, Monroe, and Adam (24) conducted a laboratory evaluation that analyzed the effects of crushed particles in asphalt concrete mixtures. Mixtures at various percentages of crushed material were evaluated. Laboratory testing included the Marshall stability, indirect tensile, resilient modulus, and creep tests. Increased percentages of crushed material yielded a substantial increase in stability. Resilient modulus data did not correlate with the percent of crushed particles or indicate resistance to rutting. Data from the creep test indicated rutting potential was very dependent on the percent of crushed aggregate.

Marker (23) stated that natural sands or uncrushed aggregate passing the No. 4 sieve was the most important factor contributing to tenderness of an asphalt concrete mixture. Most tender pavements have an excess of middle-sized sand particles in the aggregate gradation. A hump in the grading curve that has the sieve sizes raised to the 0.45 power is caused by the excess sand and occurs between the No. 4 and No. 100 sieves (11). Tenderness is most critical when this hump is near the

No. 30 sieve. This condition is generally accompanied by a relatively low amount of material passing the No. 200 sieve. Marker also stated that rounded, uncrushed aggregates are more likely to contribute to tender mixes than angular, crushed particles. This is especially true for the material passing the No. 4 sieve.

Grau (16) demonstrated in field test sections that increases in amounts of natural sand and finer sand gradations produced less stable asphalt concrete mixtures. The asphalt mixtures progressively weakened under traffic as the pavement temperatures increased. A large decrease in stability occurred when natural gravel and sand were used together. The stability values of the asphalt mixtures increased significantly when a crushed sand was used in place of natural sand.

The AASHTO Joint Task Force on rutting (2) reported that some deficiencies that have been identified as causes of rutting in asphalt concrete pavements include improper aggregate gradation and excessive use of rounded aggregates. The Task Force recommended that clean, hard and angular aggregates be used in asphalt concrete mixtures for high volume roads to help resist rutting. The FHWA Technical Advisory 5040.27 (14) recommended that natural sands be limited to 15 to 20 percent of the total weight of the aggregate for high volume roads. It was also recommended that agencies experiencing rutting problems should consider reducing the use of natural sands and incorporating more crushed fines into their mixtures.

In 1984, the Western Association of State Highway and Transportation Officials (WASHTO)(32) stated that "rutting is the most pressing issue facing highway agencies". WASHTO also stated "that state Materials

Engineers do not feel that the present procedures and specifications fully address the rutting problem. The general feeling is that the present state-of-the-art in materials testing relating to rutting needs to be upgraded through basic research".

CHAPTER III

DISCUSSION AND DESCRIPTION OF TESTING EQUIPMENT AND PROCEDURES

Several types of testing equipment and test procedures were used to determine the effects of natural sands on the engineering properties of asphalt concrete. Current state-of-the-art testing equipment was used in addition to standard laboratory equipment and procedures generally used to conduct Marshall mix designs. This more complex testing equipment and sophisticated testing procedures included the Corps of Engineers Gyratory Testing Machine (GTM), Automated Data Acquisition Testing (ADAT) System, indirect tensile test, resilient modulus test, and unconfined creep-rebound test. The laboratory equipment and test procedures used in this study are described and discussed in the following paragraphs.

Gyratory Testing Machine

Compaction of asphalt concrete materials using gyratory method applies normal forces to both the top and bottom faces of the material confined in cylindrically-shaped molds. Normal forces at designated pressures are supplemented with a kneading action or gyratory motion to compact the asphalt concrete material into a denser configuration while totally confined. The U.S. Army Corps of Engineers has developed a method, procedure, and equipment using this compaction procedure (13, 25,26).

The gyratory compaction method involves placing asphalt concrete material into a 4-inch-diameter mold and loading into the GTM at a prescribed normal stress level which represents anticipated traffic contact pressure. The asphalt material and mold are then rotated through a 1-degree gyration angle for a specified number of revolutions of the roller assembly. Figure 2 is a schematic of the gyratory compaction process. Military Standard 620 A Method 102 has correlated equivalent types of compaction and compactive efforts (12).

Gyratory Compaction	Marshall Impact Compaction
100 psi, 1-degree, 30 revolutions	50 blow per side
200 psi, 1-degree, 30 revolutions	75 blow per side

A Model 4C Gyratory Testing Machine (GTM) was used to compact all laboratory specimens in the natural sand laboratory study. Previous research with the GTM has suggested that the laboratory tests will simulate field behavior and performance under traffic when asphalt mixtures are compacted at stress levels similar to anticipated field traffic conditions (21,28). The gyratory compactive effort used in this laboratory evaluation followed the standard guidance in Military Standard 620A for the 75-blow compactive effort. The gyratory compactive effort was set at the 200 psi normal stress level, 1-degree gyration angle, and 30 revolutions of the roller assembly. The asphalt concrete specimens produced with this compactive effort satisfied the Marshall specimen dimensions of 4 inches in diameter and 2 1/2 inches thick. Figure 3 shows the WES Model 4C GTM.

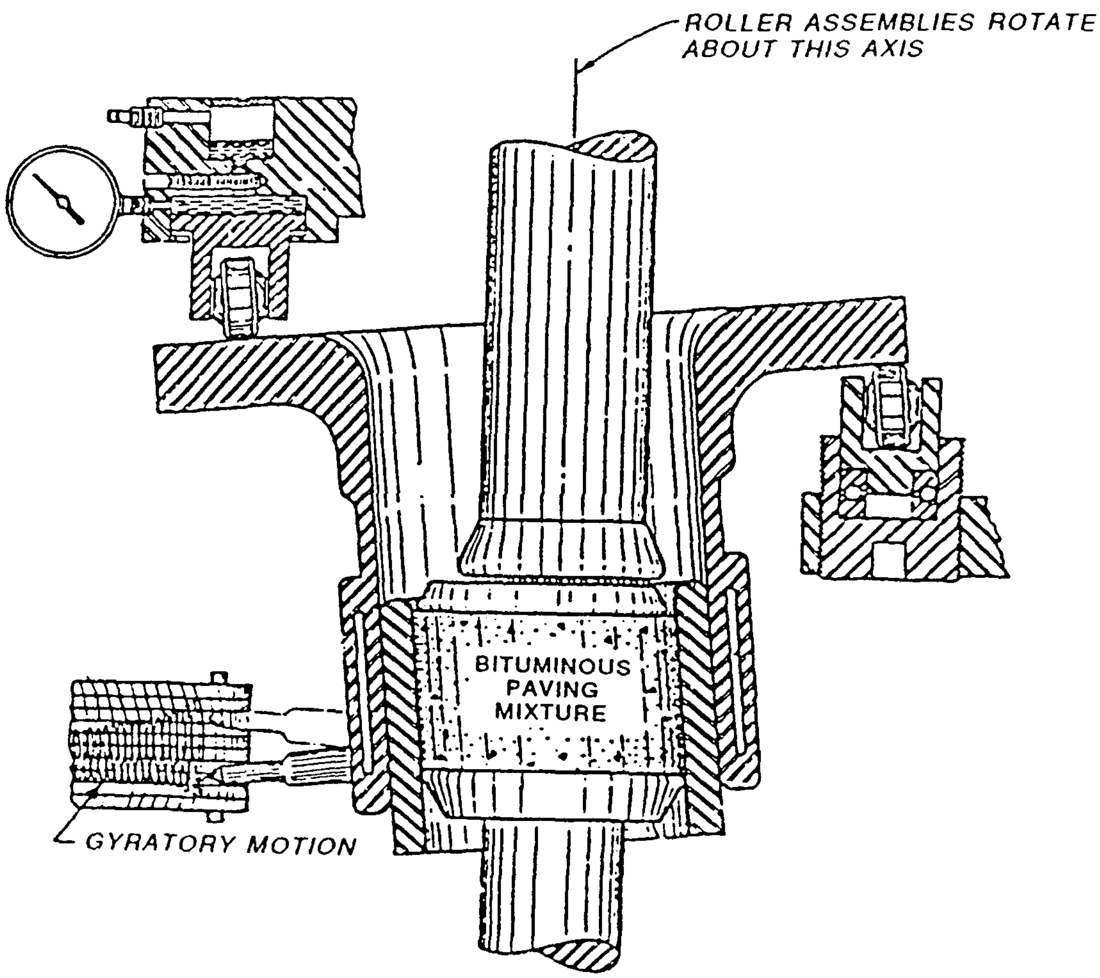
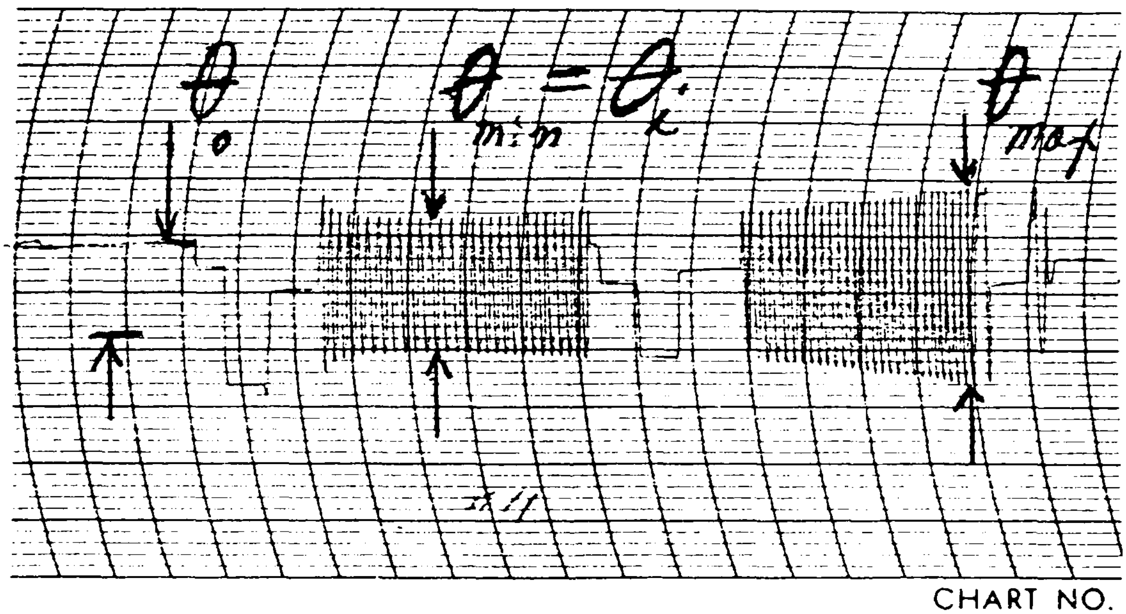


Figure 2. Schematic of Gyratory Compaction Process

Figure 3. WES Model 4C Gyratory Testing Machine

The gyratory compaction method using the GTM produces a gyratory graph or gyrograph that can be used to evaluate the asphalt concrete mixture behavior during compaction. The gyrograph indicates the relative stability behavior of the mixture during the compactive effort. The gyrograph indicates an unstable mixture when the gyrograph spreads or widens. A gyrograph that does not spread is considered stable under that loading condition (25,26).

The gyrograph can be used to produce two indices that describe the relative stability of an asphalt concrete mixture. The ratio of the final width to the intermediate width of the gyrograph is called the Gyratory Stability Index (GSI). A GSI value greater than 1.0 indicates an unstable mixture with a high asphalt content. The ratio of the intermediate width to the initial width is called the Gyratory Elasto-Plastic Index (GEPI). The GEPI value is an indicator of the quality of the aggregate. Figure 4 displays a typical gyrograph of a compacted asphalt concrete specimen.



θ_0 = Initial gyratory angle (divisions)

θ_i = Intermediate gyrograph width

θ_{max} = Maximum gyrograph width

Figure 4. Typical Gyrograph (after McRae, 1965)

Automated Data Acquisition Testing System

Previous research studies conducted in the Materials Research and Construction Technology Branch, Geotechnical Laboratory, had required accurately controlled laboratory testing and data acquisition (4). A state-of-the-art computer-operated system was assembled to conduct modern, complex asphalt concrete mixture tests. This customed-designed computer-testing system is called the Automated Data Acquisition Testing (ADAT) System. The ADAT System was specifically designed and organized to conduct three asphalt concrete mixture tests; indirect tensile, resilient modulus, and unconfined creep-rebound. Figure 5 is an overall view of the ADAT System.

The MTS electrohydraulic closed-looped material system is the main component of the ADAT System. The loading sequences of the electro-hydraulic system are controlled by an arbitrary waveform generator. The test loads are recorded by electronic load cells and the specimen deformations are measured by electronic linear variable differential transformers (LVDT). The ADAT System also includes electronic temperature control of the enclosed environmental chamber and real time color graphics.

The ADAT System is controlled by a 16-bit mini-computer designed to operate as the system's principal measurement and control station. Customized computer programs were developed to control the mechanics, monitoring systems, test data manipulations, and data storage for indirect tensile, resilient modulus and unconfined creep-rebound tests. These programs were designed to reduce operator dependency and to allow the computer to be the single system control.

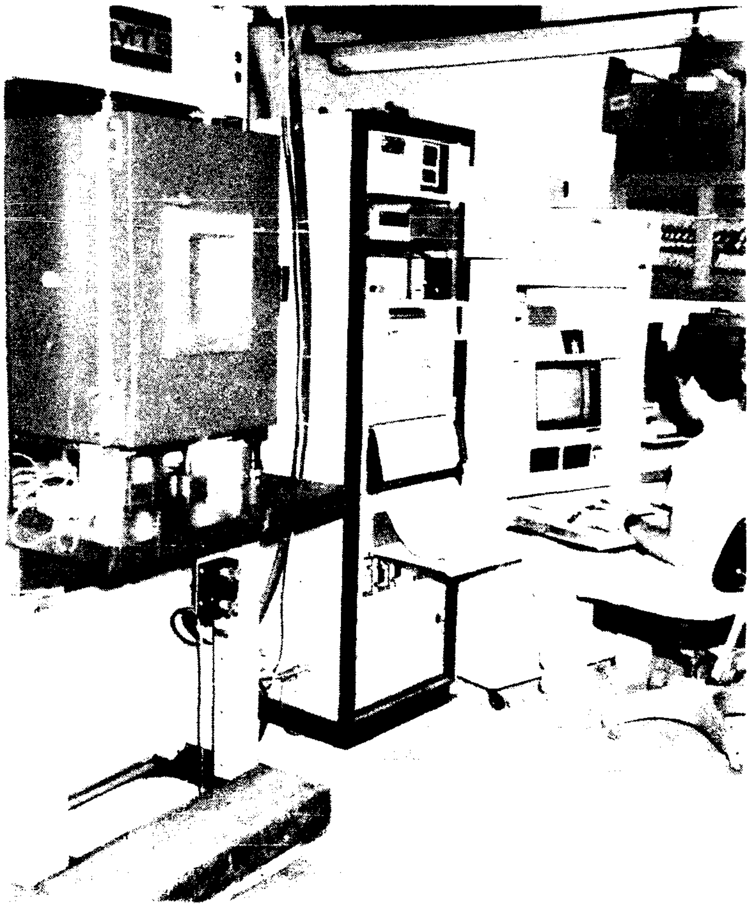


Figure 5. Overall View of Automated Data Acquisition Testing System

Indirect Tensile

Researchers in Brazil and Japan developed a testing procedure in 1953 to indirectly determine tensile strengths of materials (1). The indirect tensile test involves placing a cylinder of material horizontally between two loading plates and loading the specimen across its diameter until failure. This test procedure has been used to test soils, concrete, and asphalt concrete materials, and has been used by engineers to compute fundamental properties of materials. Figure 6 shows a schematic of the indirect tensile test.

ASTM Method D4123 provides guidance on indirect tensile testing of asphalt concrete mixtures (3). This test procedure was conducted on specimens produced at the optimum asphalt content for each aggregate blend. This test procedure is considered straight forward and generally produces consistent results. The indirect tensile test was conducted on specimens at two test temperatures, 77^0F and 104^0F. These specimens were cured in an oven at the appropriate temperature for 24 hours before testing in the environmental chamber of the ADAT System.

The indirect tensile test required that the specimens be positioned so that the loading plates were centered and the load was applied across the diameter of the specimen. The vertical load was applied at a constant deformation rate of 2 inches per minute until failure. The ultimate load was recorded at failure by the ADAT System and used to calculate the tensile strength. This testing procedure was conducted on a minimum of three specimens for each of the seven aggregate blends at both temperatures. Figure 7 shows the indirect tensile test.

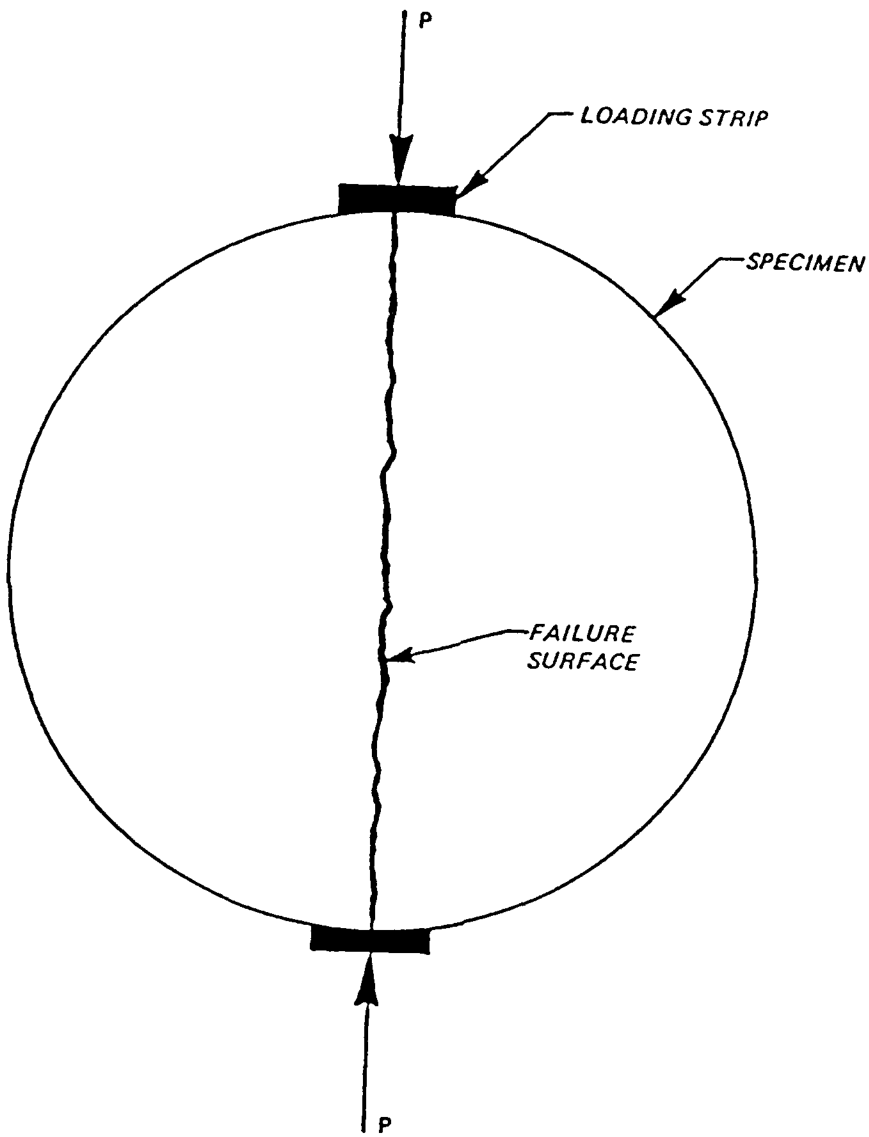


Figure 6. Schematic of Indirect Tensile Test

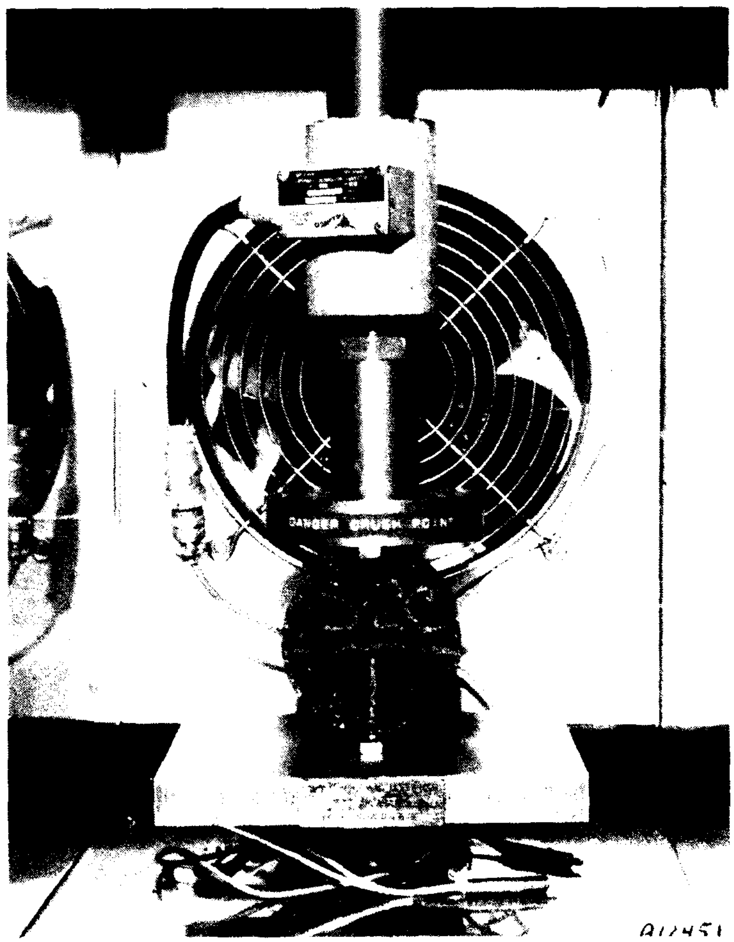


Figure 7. Indirect Tensile Test

The tensile strength was calculated using the formulation provided in ASTM D4123, as follows:

$$\text{Tensile strength} = 2P/\pi tD$$

where

P = ultimate load required to fail specimen (lb)

t = thickness of specimen (in)

D = diameter of specimen (in)

The results of the indirect tensile tests are presented and discussed in Chapter V.

Resilient Modulus

The resilient modulus test is used to evaluate the relative quality of asphalt concrete mixtures. The resilient modulus test procedure was conducted according to ASTM Method D4123 (3). Higher resilient modulus values indicate that the asphalt mixture has a greater resistance to permanent elastic deformation. This test procedure also evaluates the effects of repeated loads on asphalt concrete mixtures. The resilient modulus test is considered a nondestructive test and allows the same specimen to be tested several times.

The resilient modulus test requires the specimens to be pre-conditioned at the desired testing temperature for 24 hours. The specimens are then positioned between the loading plates in the same manner as the indirect tensile test. Horizontal and vertical deformations are measured during the loading operation with LVDTs. Figure 8 shows the resilient modulus test.

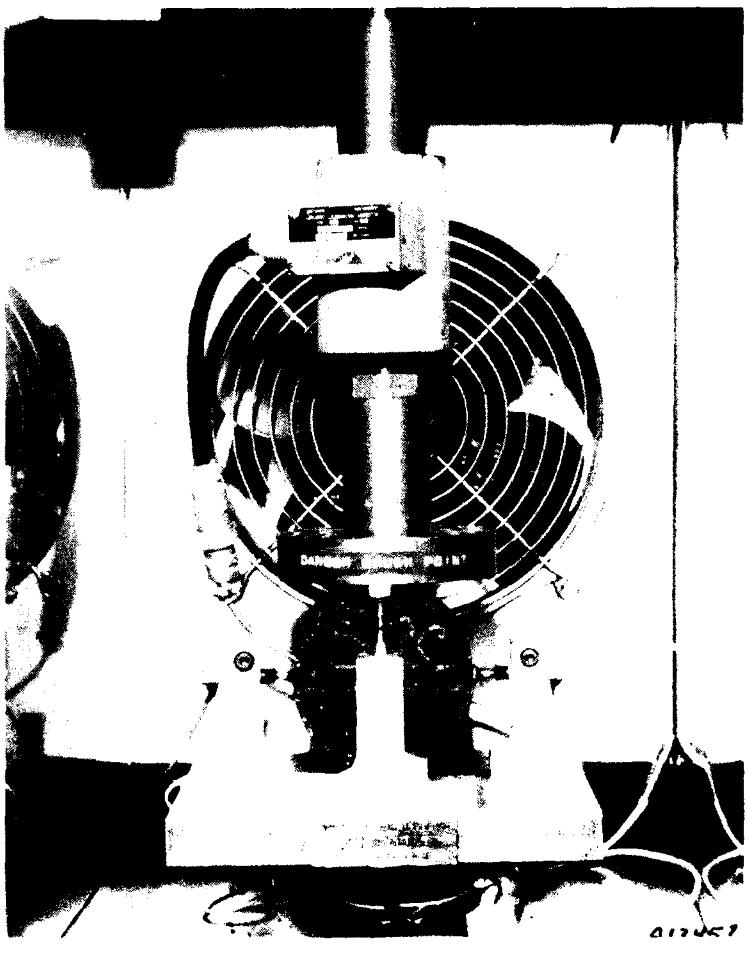


Figure 8. Resilient Modulus Test

The actual resilient modulus testing procedure for this study involved the following: the specimens were preconditioned by applying a repeated haversine waveform at a reduced load to obtain a uniform deformation readout; the magnitude of the load applied was 5 to 25 percent of the aggregate blend's tensile strength; the time of loading was set at 0.1 seconds (representative time for actual pavement loadings); the loading frequency was set at 1.0 Hz or 1 cycle per second; and the haversine waveform was applied by the arbitrary waveform generator as recommended by ASTM.

The resilient modulus test was conducted on a minimum of six specimens from each aggregate blend. Each specimen was tested in two positions, the initial position (0 degrees) and a rotated position 90 degrees from the initial position. Conducting the resilient modulus test in this manner allowed a total of twelve resilient modulus values to be determined. This procedure was conducted at both testing temperatures, 77^{0}F and 104^{0}F.

The resilient modulus value was calculated using a modified version of the equation presented in ASTM D4123. The equation used in this study assumed a Poisson's ratio of 0.35. The ASTM method suggests an equation that uses a Poisson's ratio that is calculated with horizontal and vertical deformations. The variability in the measured vertical deformation causes an inconsistency in the calculated resilient modulus value, thus producing unreliable data (7).

The resilient modulus value was calculated as follows:

$$E_{RT} = 0.62P/t\ \Delta H_T \tag{4}$$

where

E_{RT} = total resilient modulus of elasticity (psi)

P = applied repeated load (lb)

t = thickness of specimen (in)

ΔH_T = total recoverable horizontal deformation (in)

The results of the resilient modulus tests are presented and discussed in Chapter V.

Unconfined Creep-Rebound

The unconfined creep-rebound test used to evaluate the natural sand aggregate blends was developed at WES (4). This test method has no nationally recognized test procedure. The unconfined creep-rebound test was developed to evaluate the asphalt mixture's resistance to permanent deformation under severe loads. This laboratory test is one of the best indicators of rutting potential. The rebound portion of the test procedure evaluates the reaction of the asphalt concrete after severe loading.

The unconfined creep-rebound tests were performed on three Marshall specimens stacked on top of each other. These specimens were approximately 7 1/2 inches tall. The specimens were placed in the environmental chamber between the loading plates after curing in the oven for 24 hours. The loading plates were precoated with silicone grease to minimize the effect of end restraint. Two vertical LVDTs were mounted on the center specimen to record the vertical deformation during the loading and unloading phases. An average of the two readings were used to make the creep-rebound calculations. Each stack of specimens was preconditioned with a 50-pound preload, approximately a 4 psi vertical stress, before the actual testing began. Figure 9 shows the unconfined creep-rebound test.

The creep portion of the test applied a constant load for 60 minutes and then the load was released for 60 minutes for the rebound phase. The deformations and loads were recorded by the ADAT System at various times during the creep and rebound phases. These measurements were used to calculate stresses and strains and then converted into a creep modulus value. The unconfined creep-rebound test was conducted at 77^0F and 104^0F. The constant loads applied to the specimens ranged from 30 to 40 psi for the 77^0F tests and 10 to 15 psi for the 104^0F tests. Figure 10 displays a typical creep-rebound deflection versus time curve.

The results of the unconfined creep-rebound test can be used in several ways to evaluate asphalt concrete mixtures. The amount of deformation during the creep phase indicates the asphalt mixture's potential resistance to permanent deformation. Smaller axial deformations and lower creep deformation values indicate stable asphalt mixtures. The percent rebound or recovered deformation indicates the asphalt concrete mixture's ability to recover traffic induced deformation. High percent rebound values indicate that little deformation will actually occur. The creep modulus value indicates the asphalt concrete mixture's stiffness. High creep modulus values should indicate minimum potential permanent deformation.

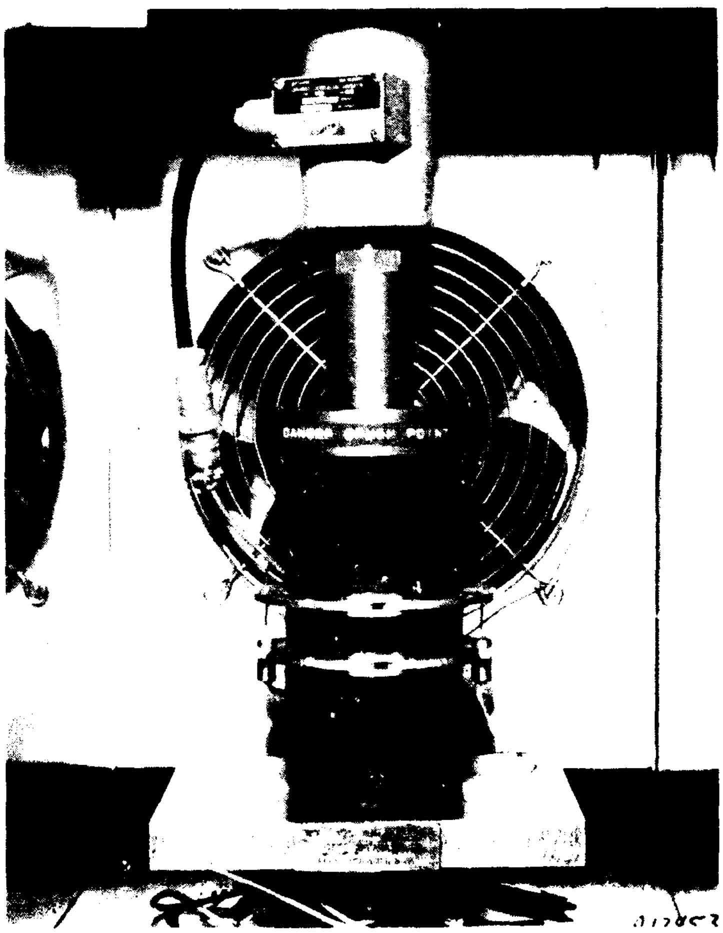


Figure 9. Unconfined Creep-Rebound Test

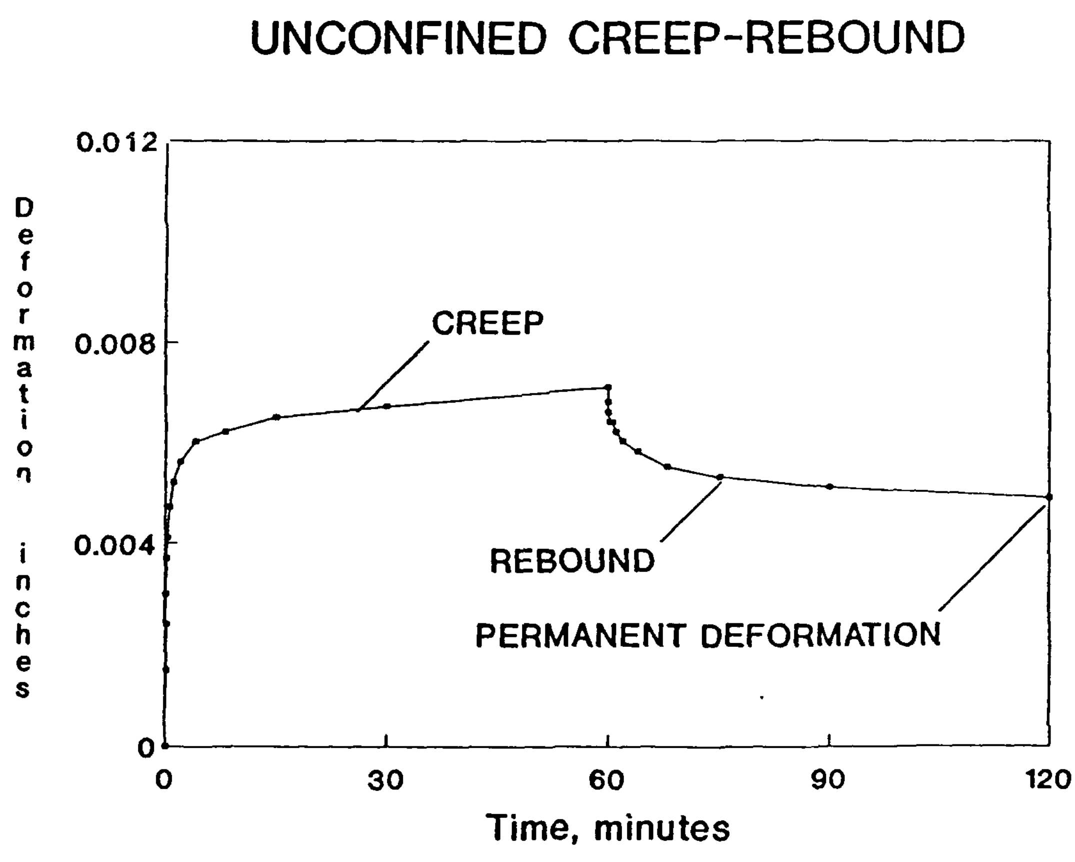


Figure 10. Typical Unconfined Creep-Rebound Curve

The creep modulus value was calculated as follows:

$$Ec - (S)(H)/D \qquad (4)$$

where

Ec = creep modulus (psi)

S = vertical stress (load/contact area; psi)

H = height of specimen (in)

D = axial deformation (in)

Test results for the creep, rebound and creep modulus values are presented and discussed in Chapter V.

CHAPTER IV

PHASE I - PRESENTATION AND ANALYSIS OF DATA

This chapter presents and discusses the results of the laboratory testing involved in Phase I of this laboratory study. Laboratory tests were conducted on the laboratory materials to determine physical properties of the asphalt cement, natural sand materials, and labstock limestone aggregate. Aggregate gradations were computed to produce aggregate blends that were as consistent as possible. Asphalt concrete mix designs were conducted for the seven aggregate blends to select the optimum asphalt contents.

Asphalt Cement

An AC-20 viscosity graded asphalt cement was selected as the asphalt material for the natural sand laboratory study. This labstock AC-20 material is generally considered a medium to hard asphalt cement. An AC-20 asphalt cement was selected because of its widespread use across the country. The AC-20 material was tested in accordance with ASTM D3381 (3) and met the requirements of Table 2 of ASTM D3381. Table 2 lists the properties of the AC-20 material.

TABLE 2

ASPHALT CEMENT PROPERTIES (ASTM D3381)

Test	Requirements[a]	Results
Viscosity - absolute, 140^0F, P	2000 $\pm$ 400	2246
Viscosity - kinematic, 275^0F, Cst	300 min	497
Penetration - 77^0F, 100g, 5 sec, 0.1 mm	60 min	80
Flash Point - Cleveland Open Cup, ^{0}F	450 min	570
Solubility in Trichloroethylene - Percent	99 min	99.94
Test on Residue from Thin Film Oven Test		
Percent Weight Loss	-----	0.21
Viscosity - 140^0F, P	10,000 max	5287
Penetration - 77^0F, 100 g, 5 sec, 0.1 mm	-----	47
Ductility - 77^0F, 5 cm/min, cm	50 min	69.5

[a] Table 2 of ASTM D3381

Natural Sand Materials

Natural sand material is generally considered to be an aggregate that has occurred naturally without any blasting or crushing. A natural sand is generally a siliceous material that has a smooth, rounded surface and is in the size range between the No. 4 and No. 200 sieves. Natural sands can be classified as a fine sand (No. 40 to No. 200), medium sand (No. 10 to No. 40) and coarse sand (No. 4 to No. 10).

Natural sand materials are often used in asphalt concrete mixtures because of the low cost and the accessibility of these materials. Two locally available natural sand materials were selected for this laboratory study. These materials were called mason sand and concrete sand. Both of these materials are typical aggregates that are used in asphalt concrete mixtures. The mason sand was a medium sand with an apparent specific gravity of 2.65 and a water absorption of 0.07 percent. The concrete sand was also a medium sand with an apparent specific gravity of 2.64 and a water absorption of 0.20 percent. Table 3 lists the aggregate gradations of the mason sand and concrete sand.

TABLE 3

AGGREGATE GRADATIONS FOR NATURAL SANDS

U.S. Standard Sieve Size	Mason Sand Percent Passing	Concrete Sand Percent Passing
No. 8	100	100
No. 16	99.6	99.0
No. 30	95.6	80.3
No. 50	47.2	14.0
No. 100	2.8	2.5
No. 200	0.5	1.4

Limestone Aggregate

The crushed limestone aggregate used in this study was obtained from Vulcan Materials in Alabama. This crushed limestone material is the labstock material used in most laboratory research evaluations at WES. This material had been separated by a Gilson shaker into various sizes. This screening operation processed the material so that the aggregate was separated into nine stockpiles, one per sieve size. The limestone aggregate had an apparent specific gravity of 2.82 and a water absorption of 0.4 and 0.8 percent for the coarse and fine aggregate material, respectively. This limestone aggregate had fractured, angular faces and a rough surface texture.

Aggregate Blends

The laboratory study required that a constant aggregate gradation be used throughout the evaluation to decrease the gradation effect on the engineering properties of the asphalt mixtures. The 3/4 inch maximum aggregate size gradation for high tire pressure applications from TM 5-822-8/AFM 88-6 was selected as the target aggregate gradation (10). This aggregate gradation was used for all aggregate blends in this study.

Aggregate blends using crushed limestone and various percentages of natural sand were blended as closely as possible to the same gradation. The aggregate blends contained 0, 10, 20, and 30 percent of each of the natural sand materials. As the percentage of natural sand increased, especially at 20 and 30 percent levels, the same aggregate gradation was not obtainable. As the percentage of natural sand increased, the amount of material passing the No. 30 sieve increased. At the 30 percent level of natural sand, a definite hump occurred at the No. 30 sieve. The aggregate gradations for this laboratory study are listed in Table 4 and shown in Figures 11-16.

As previously mentioned in the literature review (11), a hump in the aggregate grading curve that has the sieve sized raised to the 0.45 power is caused by an excessive amount of natural sand. This hump in the aggregate gradation generally occurs between the No. 4 and No. 100 sieves. Asphalt mixtures that have a hump near the No. 30 sieve are most likely to be tender or unstable. The aggregate gradations for this laboratory study have been plotted on a chart that has the sieve sizes raised to the 0.45 power. These gradations are shown in Figures 17-23. It is very evident that as the percentage of natural sand increases, a hump at the No. 30 sieve develops. A slight hump is seen at 20 percent natural sand while a very distinctive hump is noticed at 30 percent natural sand. This indicates that both 20 and 30 percent sand are sensitive and tender.

TABLE 4

AGGREGATE GRADATIONS FOR NATURAL SAND LABORATORY STUDY

U.S. Standard Sieve Size	Recommended Limits	Aggregate Blends						
		S-0	S-1M	S-2M	S-3M	S-1C	S-2C	S-3C
3/4 inch	100	100	100	100	100	100	100	100
1/2 inch	82-96	88.9	88.9	88.9	88.9	88.9	88.9	88.9
3/8 inch	75-89	81.8	81.8	81.8	81.8	81.8	81.8	81.8
No. 4	59-73	66.5	66.5	66.5	66.5	66.5	66.5	66.5
No. 8	46-60	53.5	53.5	53.5	53.5	53.5	53.5	53.5
No. 16	34-48	40.8	40.8	40.7	40.7	40.7	40.6	42.3
No. 30	24-38	31.3	31.8	33.8	38.4	30.5	33.8	35.8
No. 50	15-27	22.1	23.3	22.6	23.8	21.6	19.8	15.9
No. 100	8-18	12.4	11.6	10.5	10.2	12.1	11.4	10.7
No. 200	3-6	5.8	5.4	5.5	5.9	5.5	5.4	5.7

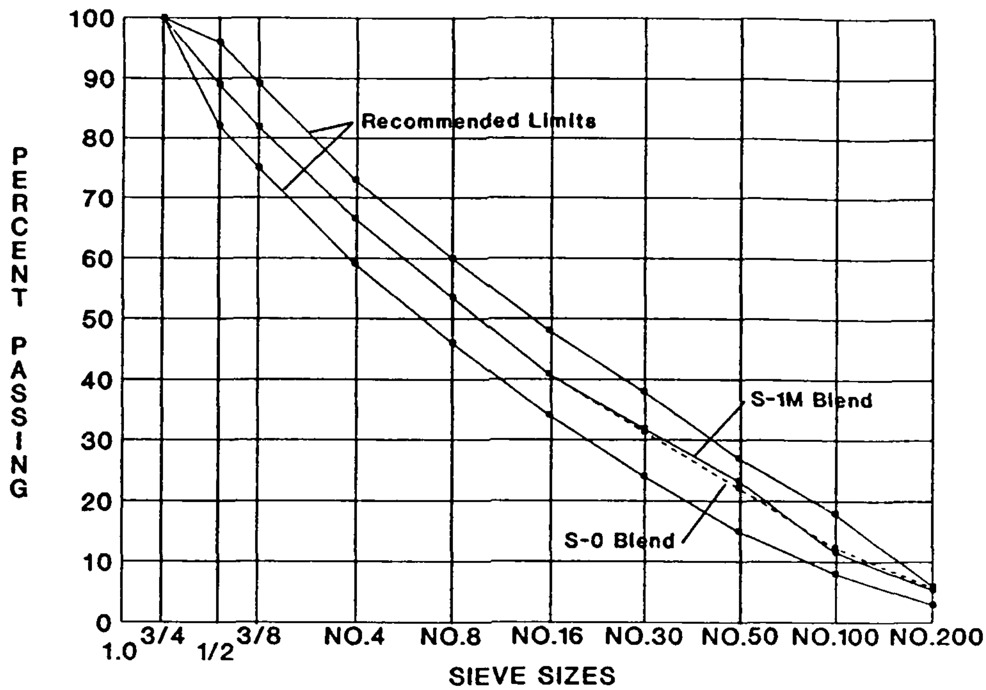


Figure 11. Aggregate Gradation Curve for S-1M Blend

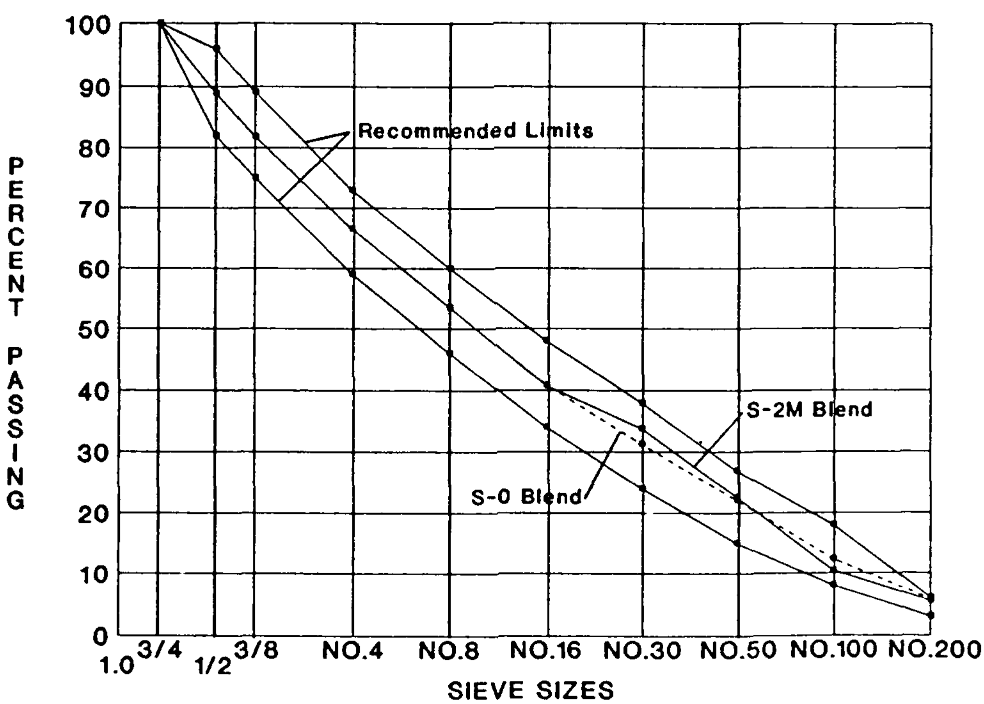


Figure 12. Aggregate Gradation Curve for S-2M Blend

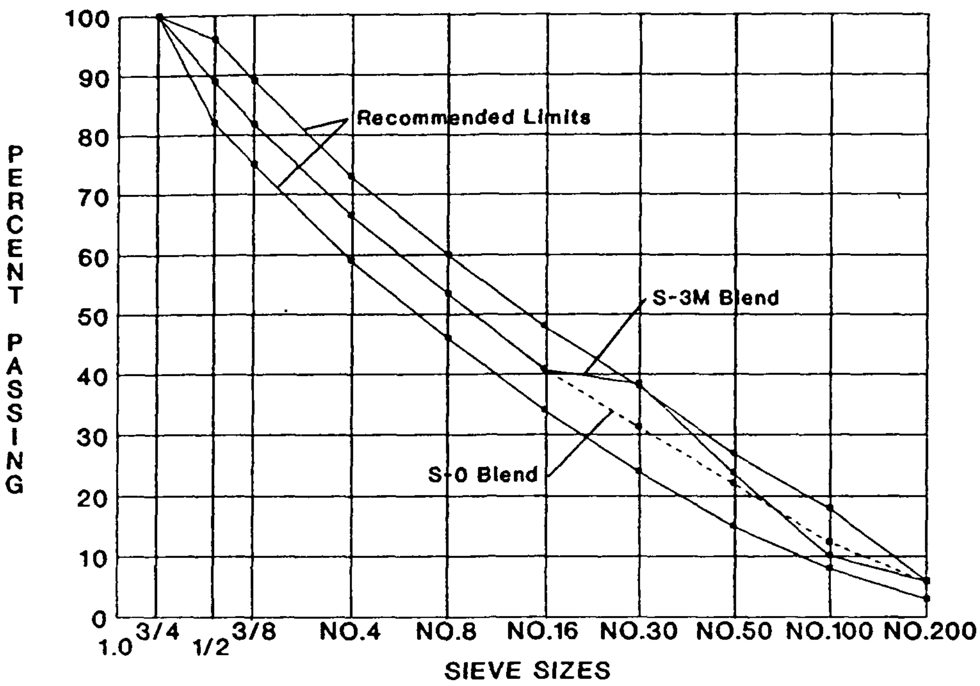


Figure 13. Aggregate Gradation Curve for S-3M Blend

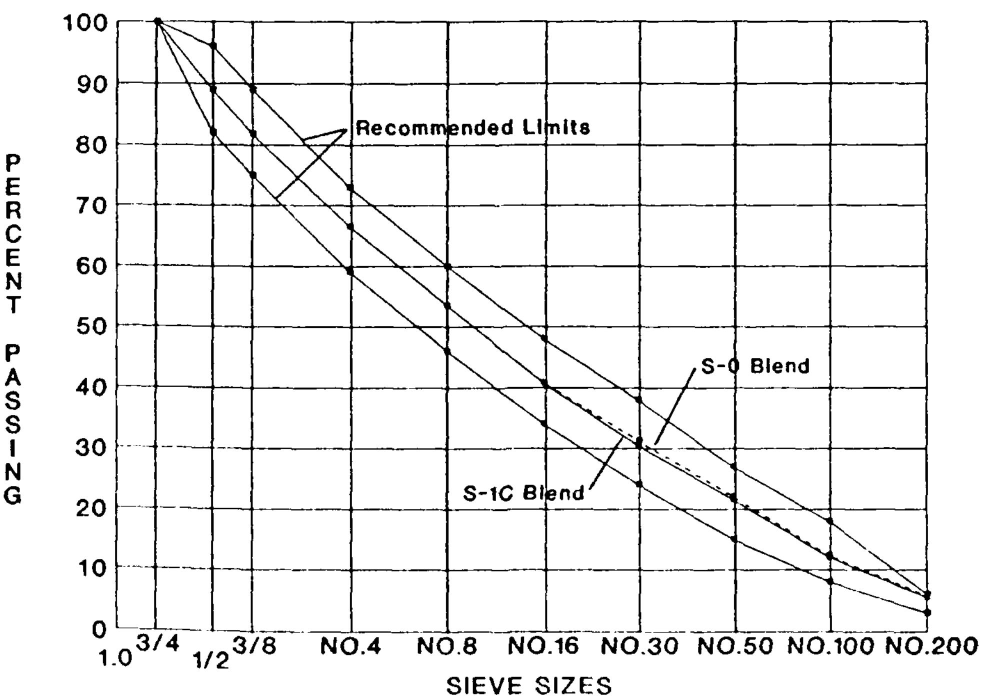


Figure 14. Aggregate Gradation Curve for S-1C blend

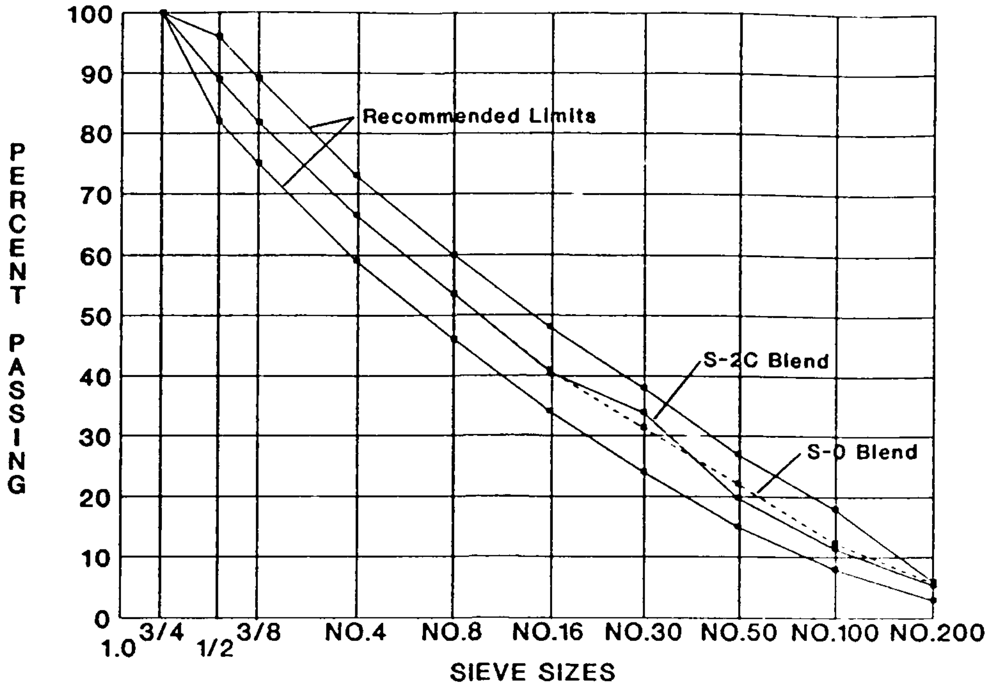


Figure 15. Aggregate Gradation Curve for S-2C Blend

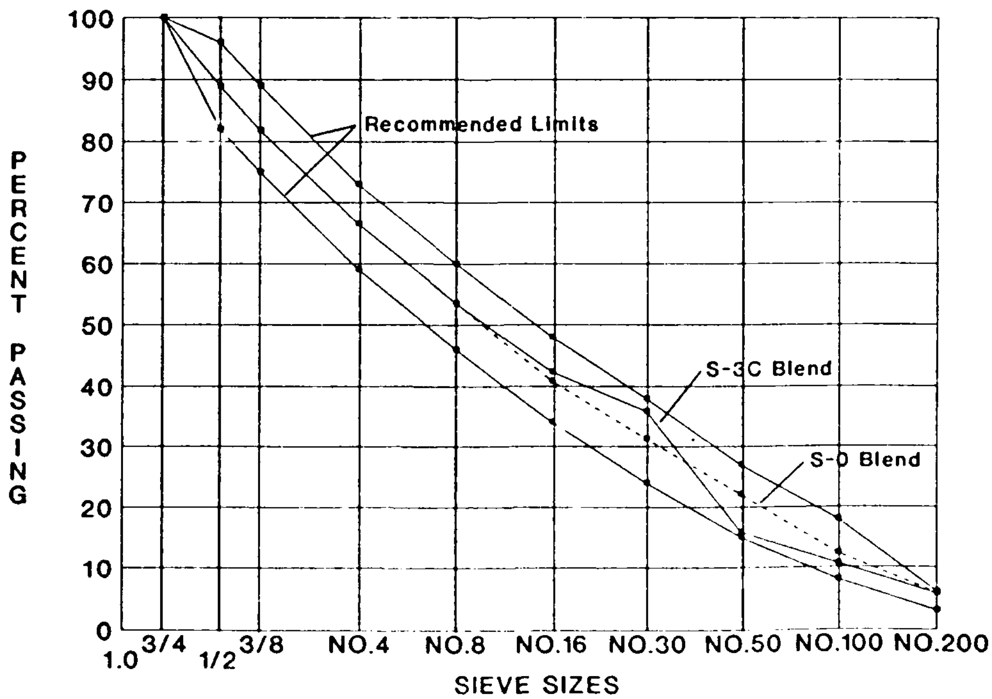


Figure 16. Aggregate Gradation Curve for S-3C Blend

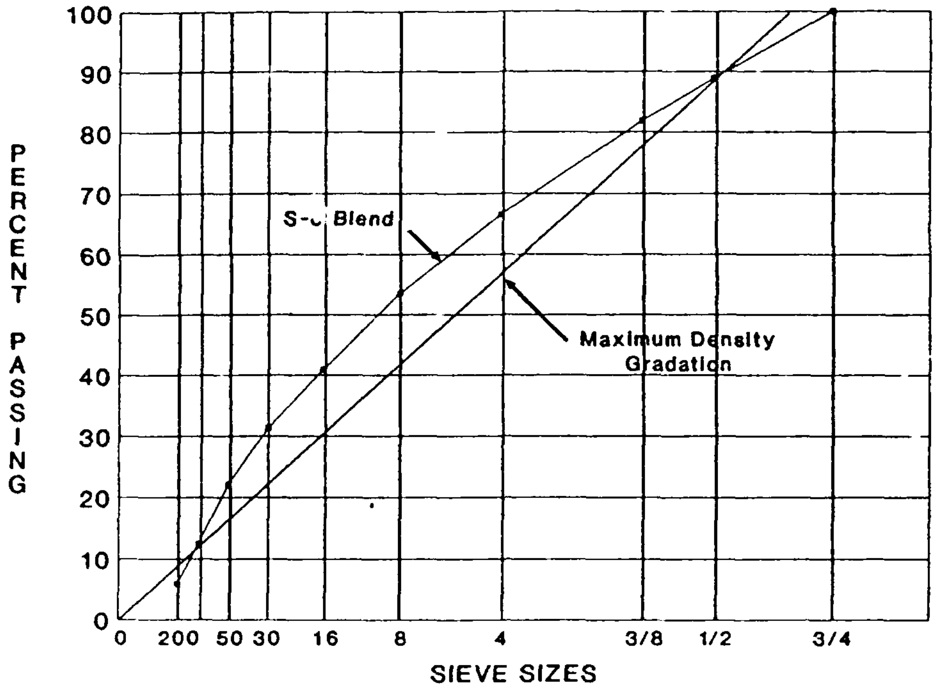


Figure 17. S-0 Gradation Curve Raised to 0.45 Power

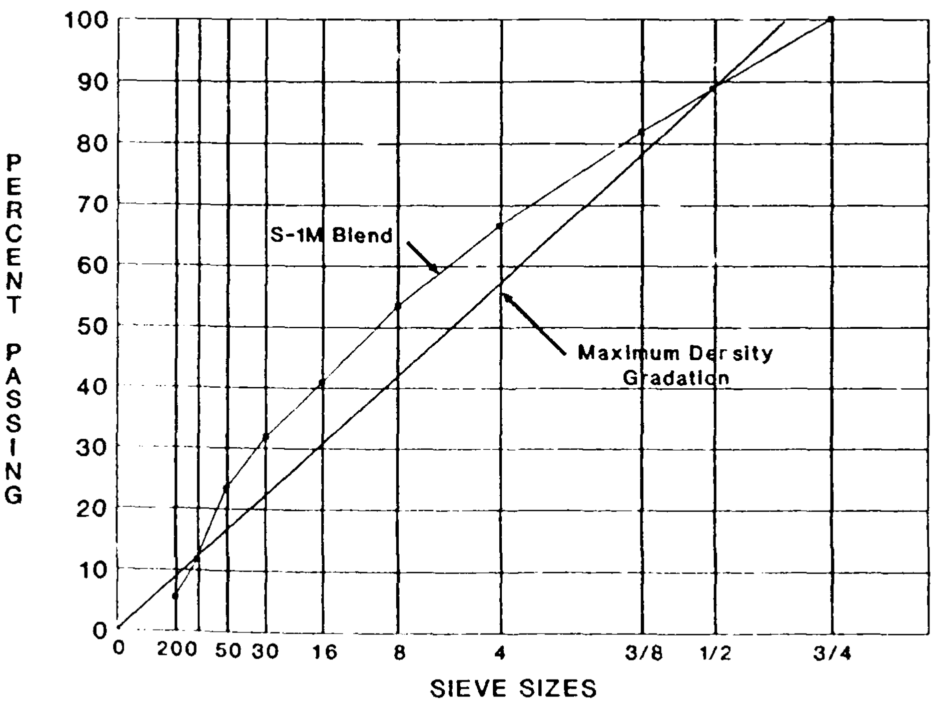


Figure 18. S-1M Gradation Curve Raised to 0.45 Power

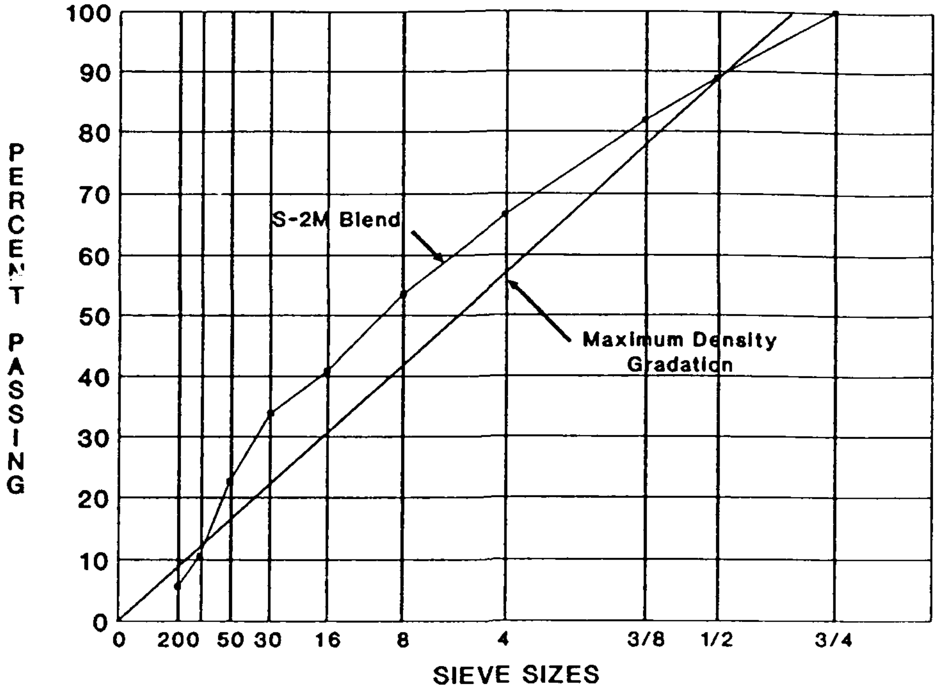


Figure 19. S-2M Gradation Curve Raised to 0.45 Power

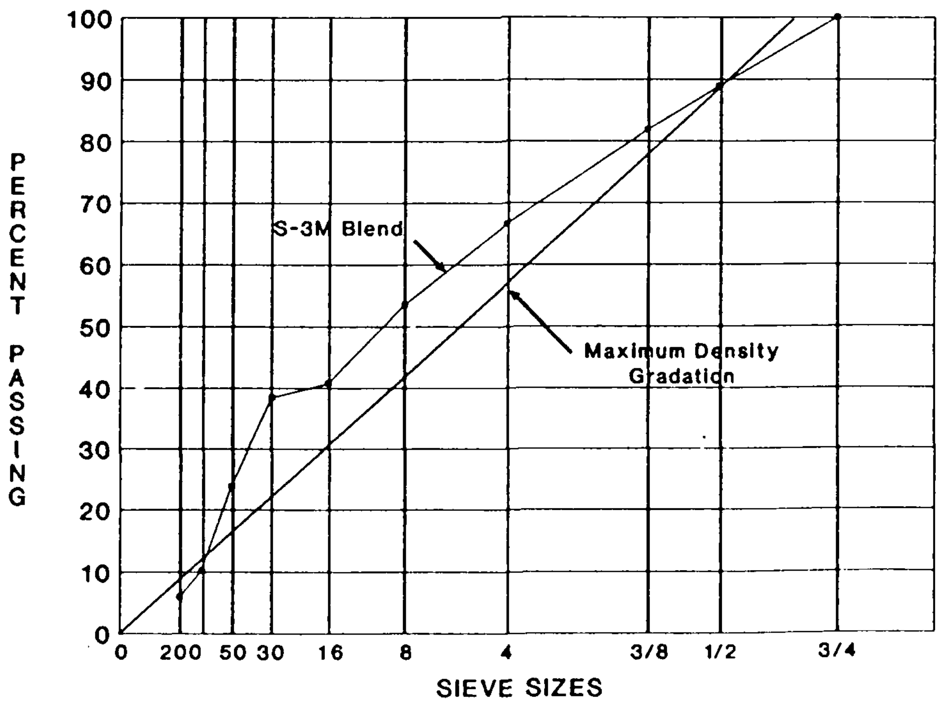


Figure 20. S-3M Gradation Curve Raised to 0.45 Power

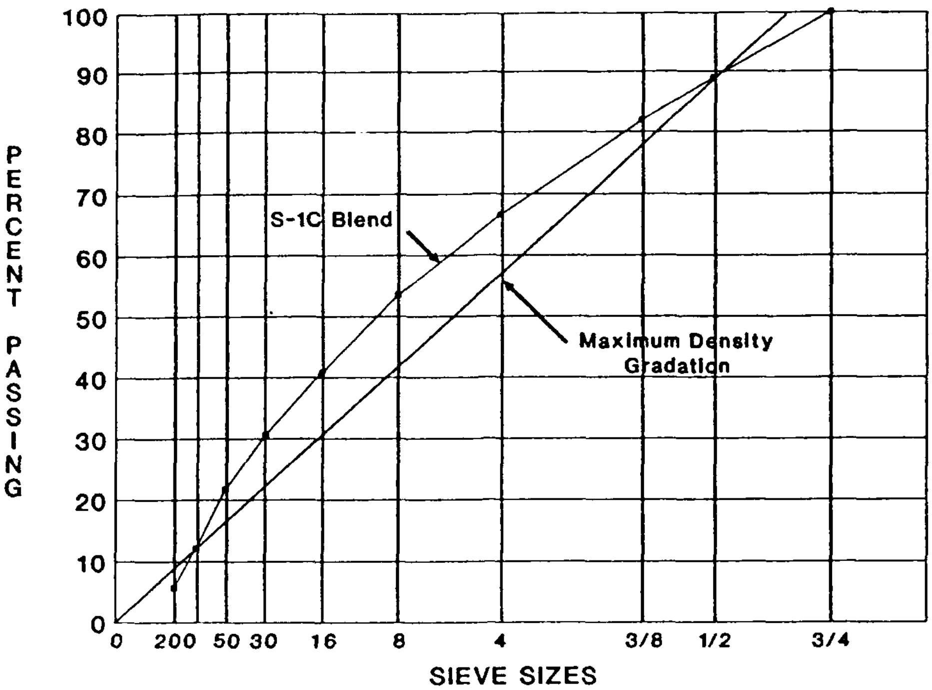


Figure 21. S-1C Gradation Curve Raised to 0.45 Power

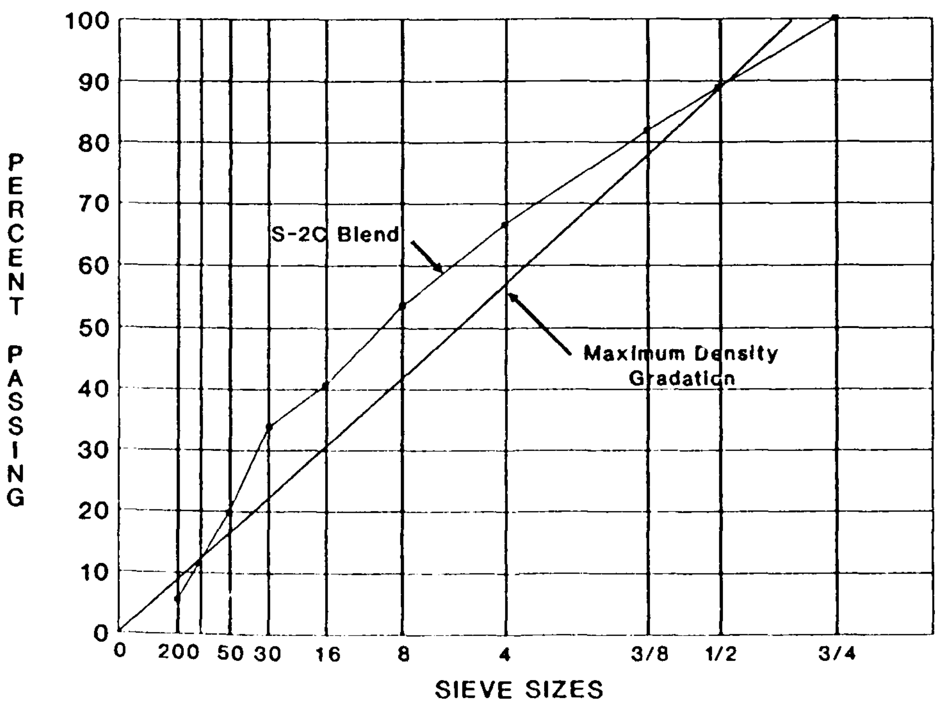


Figure 22. S-2C Gradation Curve Raised to 0.45 Power

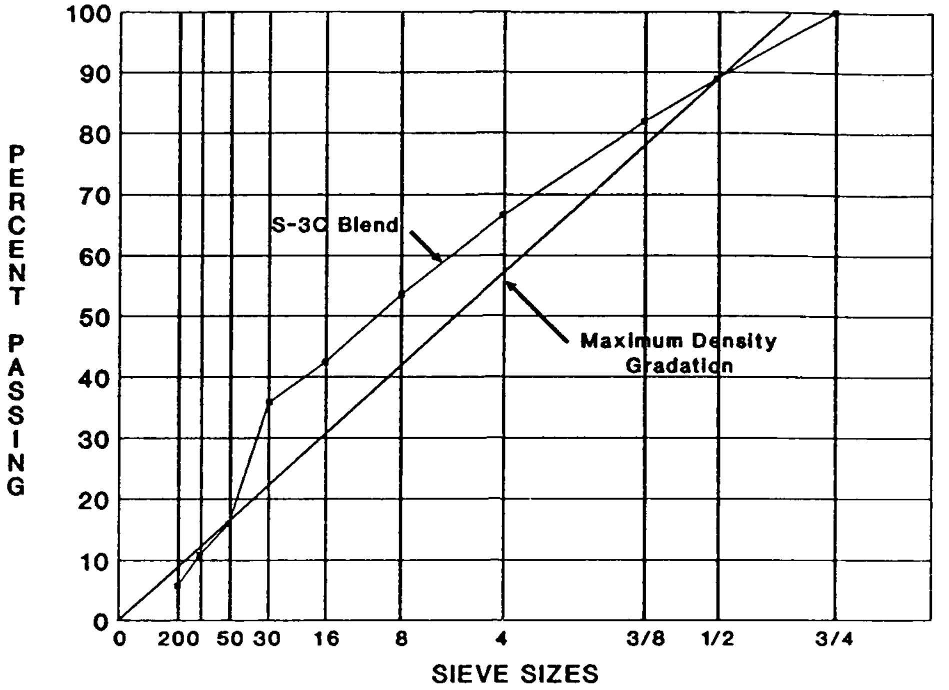


Figure 23. S-3C Gradation Curve Raised to 0.45 Power

Mix Designs

The Marshall Mix Design procedure, as outlined in Military Standard 620A (12), was used to determine optimum asphalt contents for the seven aggregate blends of this study. Each optimum asphalt content was selected at 4 percent voids total mix. The Marshall criteria normally used to determine the acceptability of the asphalt content is listed in Table 5. The optimum asphalt contents selected from these mix designs were used to produce all the specimens for Phase II.

The Gyratory Testing Machine was used to compact all specimens for the mix designs. The gyratory compactive effort used in this study was 200 psi pressure, 1-degree gyration angle, and 30 revolutions. This compaction was equivalent to a 75-blow hand hammer compactive effort that is normally used for heavy-duty pavements.

The Marshall procedure requires that compacted specimens, 4-inches in diameter and 2 1/2-inches thick, be tested with the Marshall Apparatus which is shown in Figure 24. This procedure is used to determine the stability and flow of the asphalt mixture. The stability of an asphalt mixture is an indicator of mix strength defined as the resistance to deformation under a load. The flow valve is an indicator of mix plasticity measured as the deformation at the maximum load.

The Marshall procedure requires that a range of asphalt contents be evaluated for a given aggregate gradation. Asphalt contents above and below the projected optimum asphalt content were evaluated. Data for all seven mix designs are listed in Table 6. Each value represents an average for three test specimens. The Marshall procedure also requires that mixture properties be plotted versus the asphalt content. The

TABLE 5

CRITERIA FOR DETERMINING ACCEPTABILITY OF MIXTURE

Test Property	Heavy-duty Pavement Requirement (a)
Marshall stability - lbs	1800 min
Unit weight - pcf	Not used
Flow - 0.01 inch	16 max
Voids total mix - percent	3 - 5
Voids filled with asphalt - percent	70 - 80

(a) TM 5-822-2/AFM 88-6, Chap 9

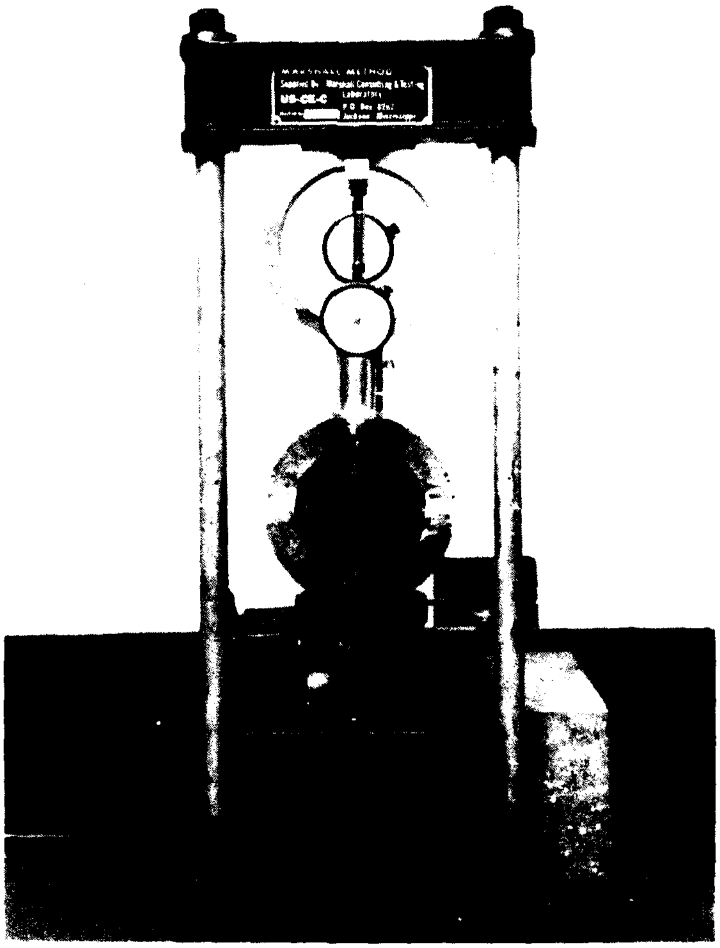


Figure 24. Marshall Apparatus

mixture properties plotted for this study were unit weight, stability, flow, voids total mix, voids filled with asphalt, and voids in mineral aggregate (VMA). The mix design plots for the seven aggregate blends are shown in Figures 25-31.

The selected optimum asphalt contents are as follows:

S-0	-	5.2 percent
S-1M	-	4.9 percent
S-2M	-	4.6 percent
S-3M	-	4.5 percent
S-1C	-	4.8 percent
S-2C	-	4.5 percent
S-3C	-	4.1 percent

TABLE 6

ASPHALT CONCRETE MIX DESIGN PROPERTIES

Aggregate Blend	Asphalt Content (percent)	Unit Weight (pcf)	Voids Total Mix (percent)	Voids in Mineral Aggregate (percent)	Voids Filled With Asphalt (percent)	Stability (lbs)	Flow (0.01 in)	Gyratory Stability Index
S-0	4.5	153 1	6.2	16.8	63.1	2061	8	0.92
	5.0	154 6	4.6	16.5	72.1	2294	9	0.92
	5.5	155.4	3.3	16.5	80.0	2519	11	0.96
	6.0	155.8	2.3	16.7	86.2	2716	10	1.00
	6.5	156.1	1.4	17.1	91.8	2589	13	1.23
	7.0	155.3	1.1	17.9	93.9	1961	16	1.64
S-1M	4.0	153.2	6.5	16.0	59.4	1786	8	0.93
	4.5	154.3	5.0	15.7	68.2	1987	8	0.93
	5.0	155.2	3.7	15.7	76.4	1955	9	0.96
	5.5	155.7	2.7	15.9	83.0	2397	9	1.00
	6.0	156.2	1.6	16.1	90 1	2453	10	1.00
	6.5	155.3	1.3	16.9	92.3	1829	15	1.52

TABLE 6 (continued)

Aggregate Blend	Asphalt Content (percent)	Unit Weight (pcf)	Voids Total Mix (percent)	Voids in Mineral Aggregate (percent)	Voids Filled With Asphalt (percent)	Stability (lbs)	Flow (0.01 in)	Gyratory Stability Index
S-2M	4.0	153.6	5.7	15.2	62.5	1805	8	0.93
	4.5	154.4	4.4	15.1	70.9	1785	9	0.93
	5.0	155.2	3.2	15.2	79.0	2019	9	1.00
	5.5	155.9	2.0	15.2	86.8	2134	10	1.07
	6.0	155.4	1.5	15.9	90.6	1674	12	1.13
	6.5	154.3	1.5	17.0	91.2	1204	17	2.00
S-3M	3.5	151.9	6.9	15.1	54.3	1494	7	0.93
	4.0	153.2	5.4	15.2	64.5	1525	8	0.96
	4.5	154.2	4.0	14.7	72.8	1566	9	0.96
	5.0	154.9	2.8	14.8	81.1	1707	9	1.00
	5.5	155.3	1.8	15.0	88.0	1719	10	1.14
	6.0	154.7	1.4	15.7	91.1	1303	14	1.44

TABLE 6 (continued)

Aggregate Blend	Asphalt Content (percent)	Unit Weight (pcf)	Voids Total Mix (percent)	Voids in Mineral Aggregate (percent)	Voids Filled With Asphalt (percent)	Stability (lbs)	Flow (0.01 in)	Gyratory Stability Index
S-1C	4.0	154.1	5.8	15.3	62.1	2113	8	0.96
	4.5	154.7	4.7	15.4	69.5	2038	8	0.96
	5.0	155.3	3.6	15.6	76.9	2143	9	1.00
	5.5	156.1	2.3	15.5	85.2	2438	9	1.00
	6.0	156.7	1.2	15.7	92.4	2347	10	1.38
	6.5	155.8	1.0	16.6	94.0	1798	14	1.40
S-2C	3.5	154.4	6.4	14.7	56.5	1547	8	0.93
	4.0	154.5	5.0	14.5	65.5	1661	8	0.93
	4.5	155.3	3.8	14.6	74.0	1897	8	0.96
	5.0	155.9	2.7	14.7	81.6	1985	9	1.00
	5.5	156.6	1.5	14.8	89.9	2069	9	1.11
	6.0	155.9	1.2	15.6	92.3	1608	14	1.30

TABLE 6 (continued)

Aggregate Blend	Asphalt Content (percent)	Unit Weight (pcf)	Voids Total Mix (percent)	Voids in Mineral Aggregate (percent)	Voids Filled With Asphalt (percent)	Stability (lbs)	Flow (0.01 in)	Gyratory Stability Index
S-3C	3.0	152.7	7.0	14.1	50.4	1320	7	0.96
	3.5	153.9	5.6	13.9	59.7	1447	8	0.96
	4.0	154.8	4.2	13.8	69.6	1491	8	1.00
	4.5	155.6	3.0	13.8	78.3	1663	8	1.00
	5.0	156.5	1.7	13.8	87.7	1835	8	1.00
	5.5	156.2	1.7	14.4	92.4	1541	13	1.37

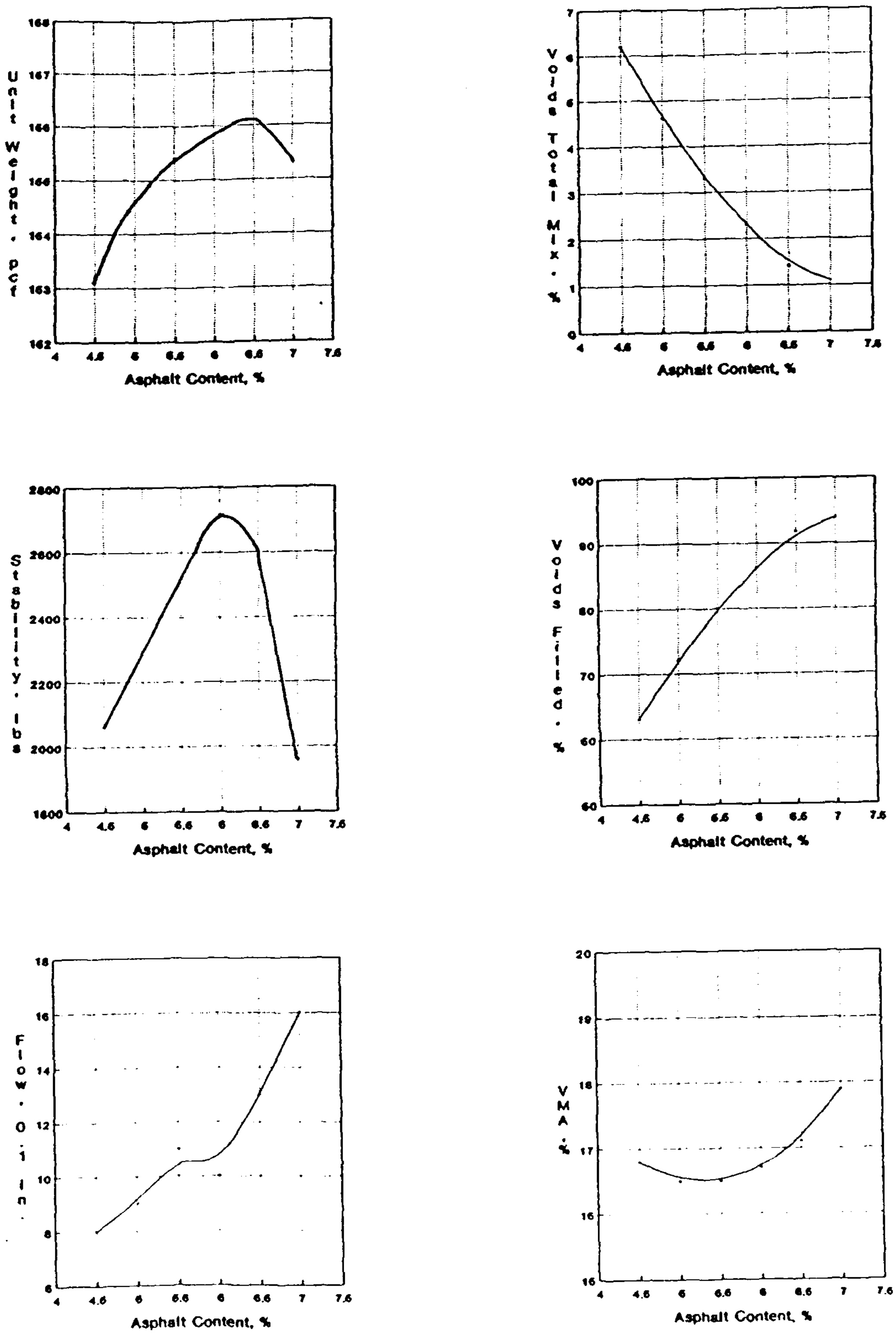


Figure 25. Mix Design Plots for S-0 Blend

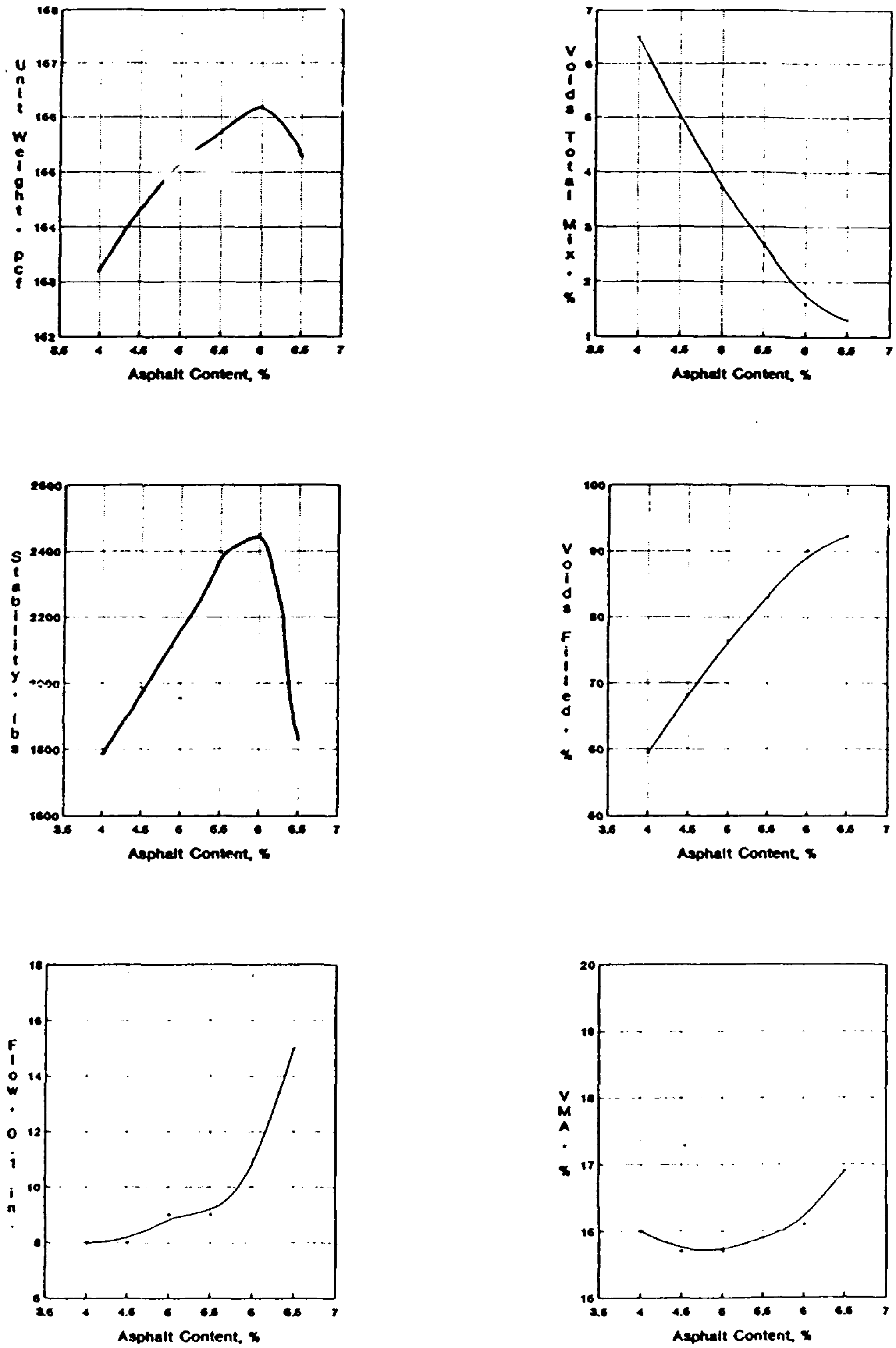


Figure 26. Mix Design Plots for S-1M Blend

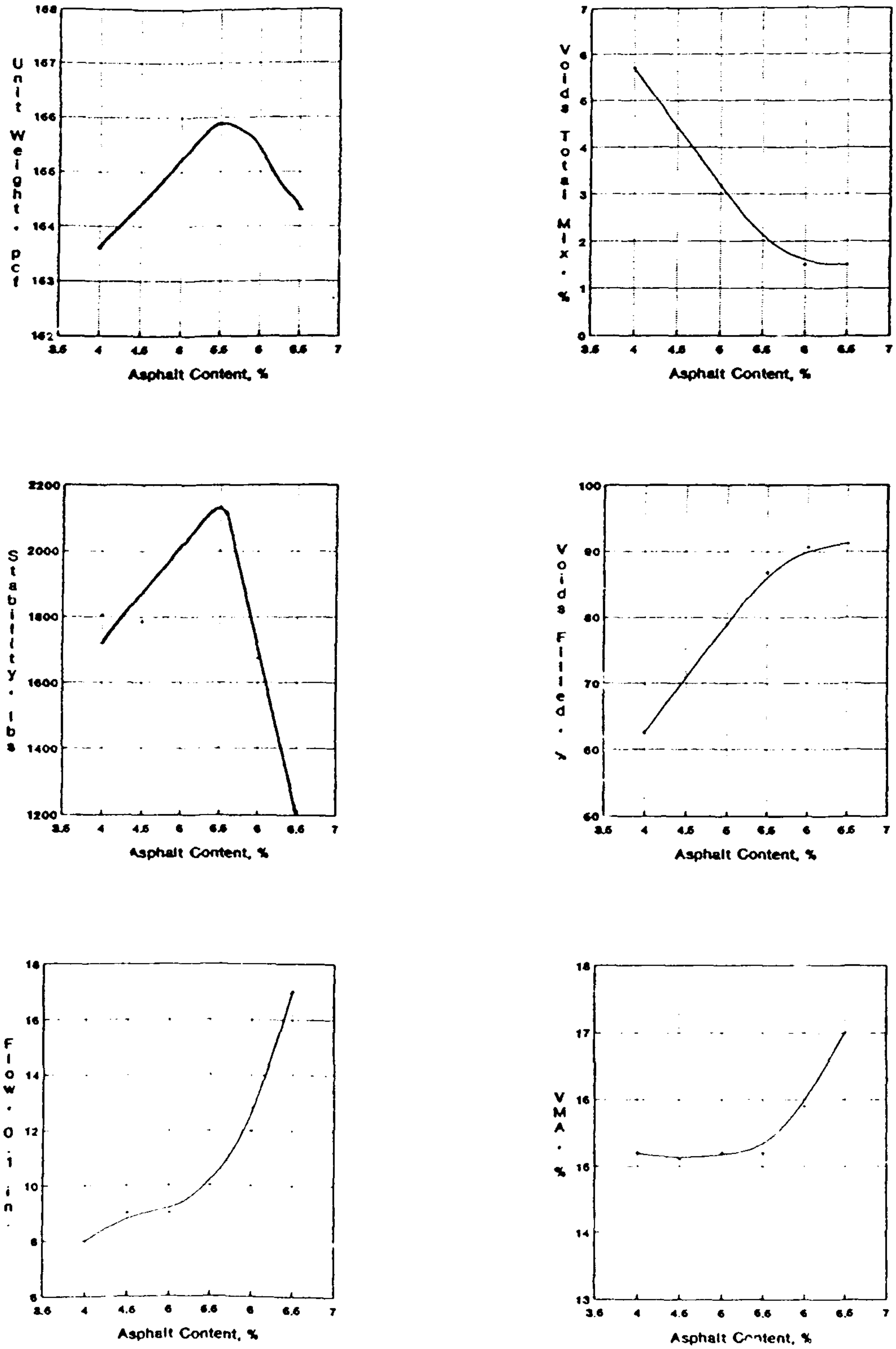


Figure 27. Mix Design Plots for S-2M Blend

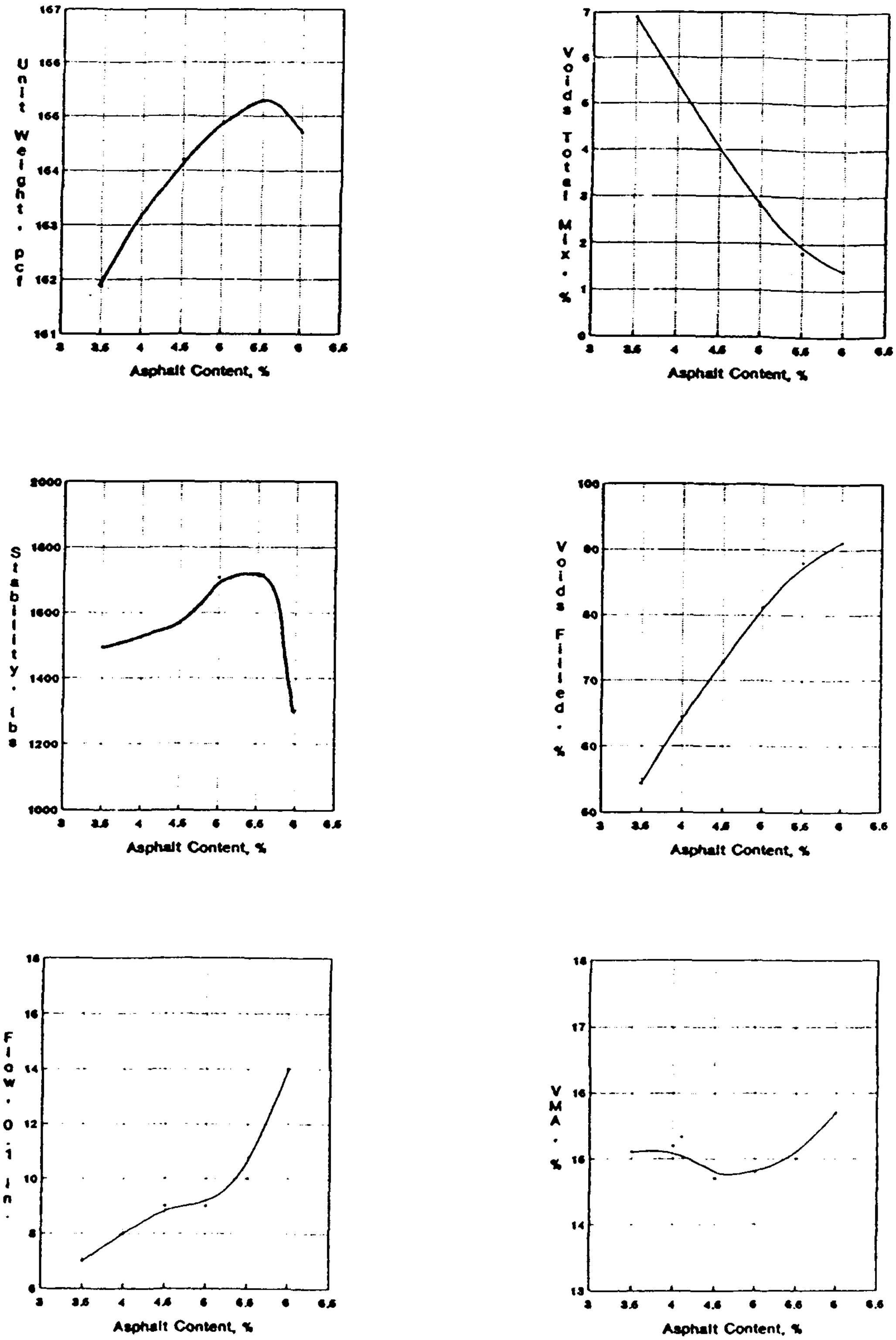


Figure 28. Mix Design Plots for S-3M Blend

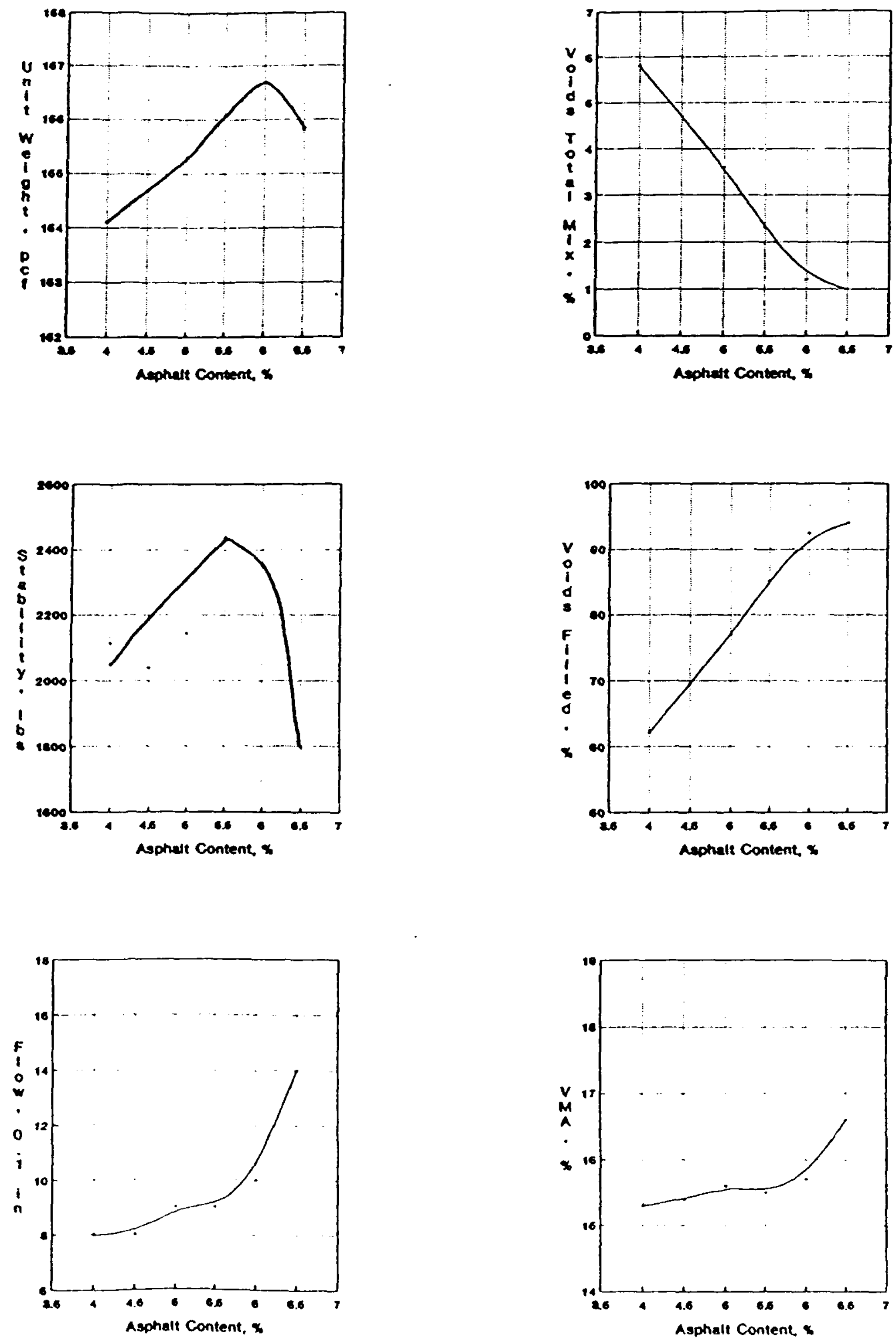


Figure 29. Mix Design Plots for S-1C Blend

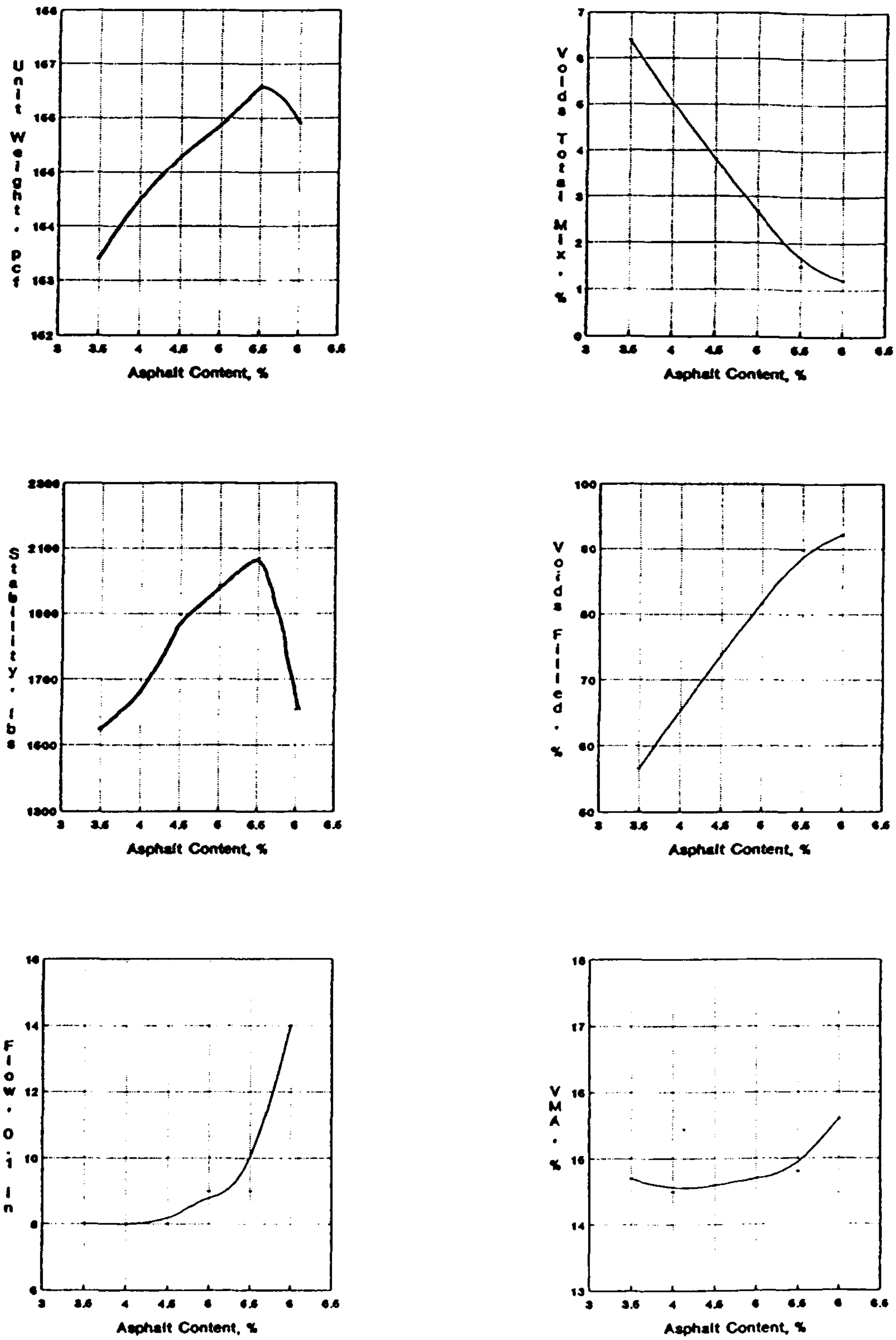


Figure 30. Mix Design Plots for S-2C Blend

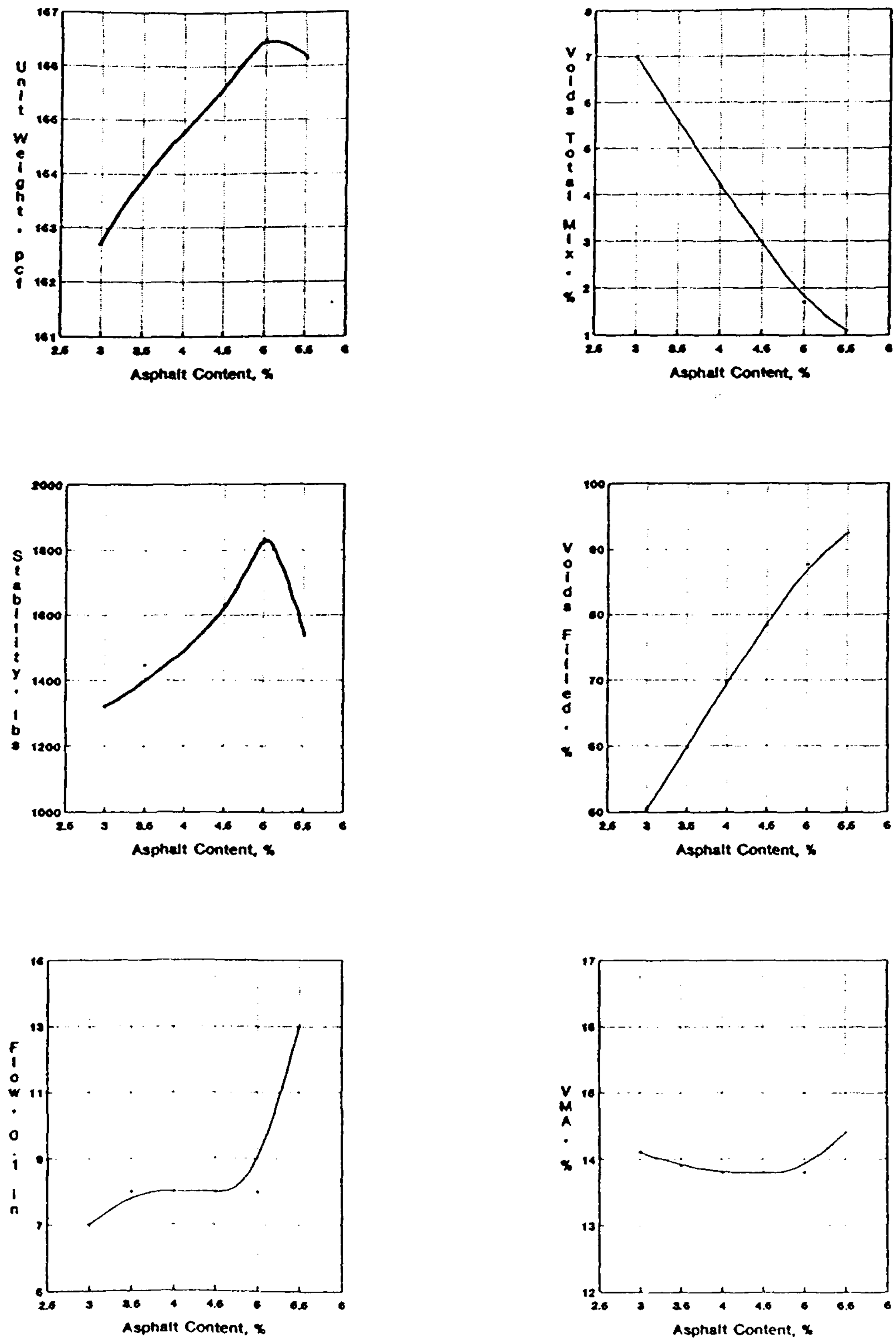


Figure 31. Mix Design Plots for S-3C Blend

Mixture Properties at Optimum Asphalt Content

Table 7 lists a summary of the mix design properties at the optimum asphalt content for each aggregate blend. Several observations and trends were observed from the mixture properties. The optimum asphalt content for each natural sand material decreased as the percentage of natural sand material increased. The optimum asphalt content for the mason sand blends decreased from 5.2 percent at 0 percent sand to 4.5 percent at 30 percent sand. The optimum asphalt content for the concrete sand blends also decreased from 5.2 percent at 0 percent sand to 4.1 percent at 30 percent sand. Figures 32-33 show the optimum asphalt content versus percent sand in mixture.

The stability value for the aggregate blends at the optimum asphalt content decreased as the percentage of natural sand increased. The stability value for the mason sand blends decreased from 2395 lbs at 0 percent sand to 1570 lbs at 30 percent sand. The stability value for the concrete sand blends decreased from 2395 lbs at 0 percent sand to 1550 lbs at 30 percent sand, a reduction in stability of approximately 35 percent. The stability values versus percent sand in mixture are shown in Figures 34-35. Another trend that was observed in the selection of optimum asphalt contents was a decrease in voids in mineral aggregate (VMA) as the percentage of natural sand increased. The VMA value for the mason sand blends decreased from 16.4 percent at 0 percent sand to 14.7 percent at 30 percent sand. The VMA value for the concrete sand blends also decreased from 16.4 percent at 0 percent sand to 13.8 percent at 30 percent sand. The VMA values versus percent sand in mixture are shown in Figures 36-37.

TABLE 7

SUMMARY OF MIX DESIGN PROPERTIES AT OPTIMUM ASPHALT CONTENT

Aggregate Blend	Optimum Asphalt Content	Unit Weight (pcf)	Stability (lbs)	Flow (0.01 in)	Voids Total Mix (percent)	Voids in Mineral Aggregate (percent)	Voids Filled With Asphalt (percent)
S-0	5.2	154.9	2395	9	4.0	16.4	75.0
S-1M	4.9	155.0	2180	8	4.0	15.7	75.0
S-2M	4.6	154.6	1880	9	4.0	15.0	73.0
S-3M	4.5	154.2	1570	9	4.0	14.7	73.0
S-1C	4.8	155.0	2200	8	4.0	15.5	74.0
S-2C	4.5	155.3	1900	8	4.0	14.8	74.0
S-3C	4.1	154.9	1550	8	4.0	13.8	72.0

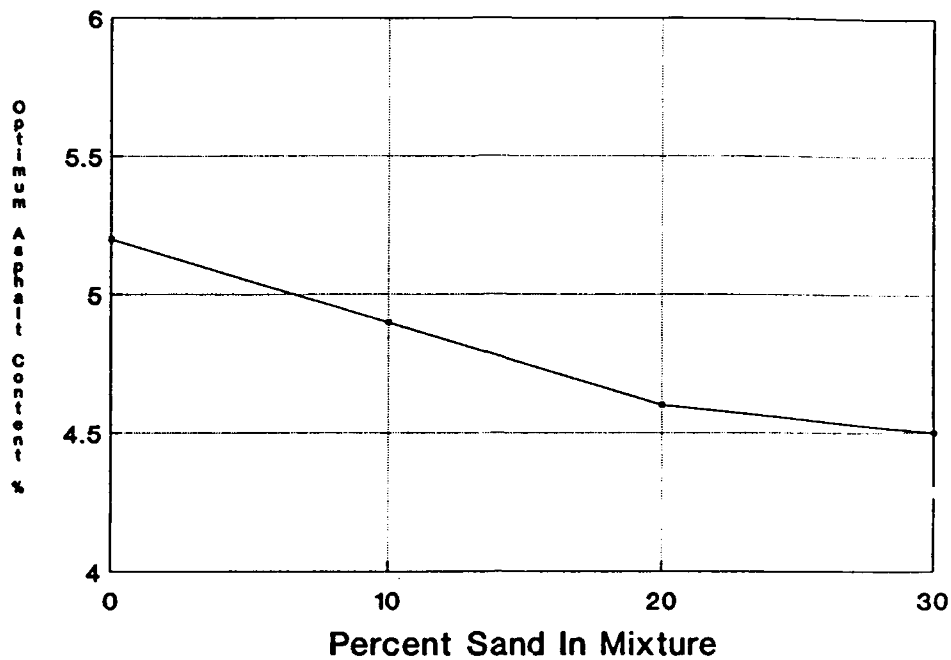


Figure 32. Optimum Asphalt Content Versus Percent Mason Sand

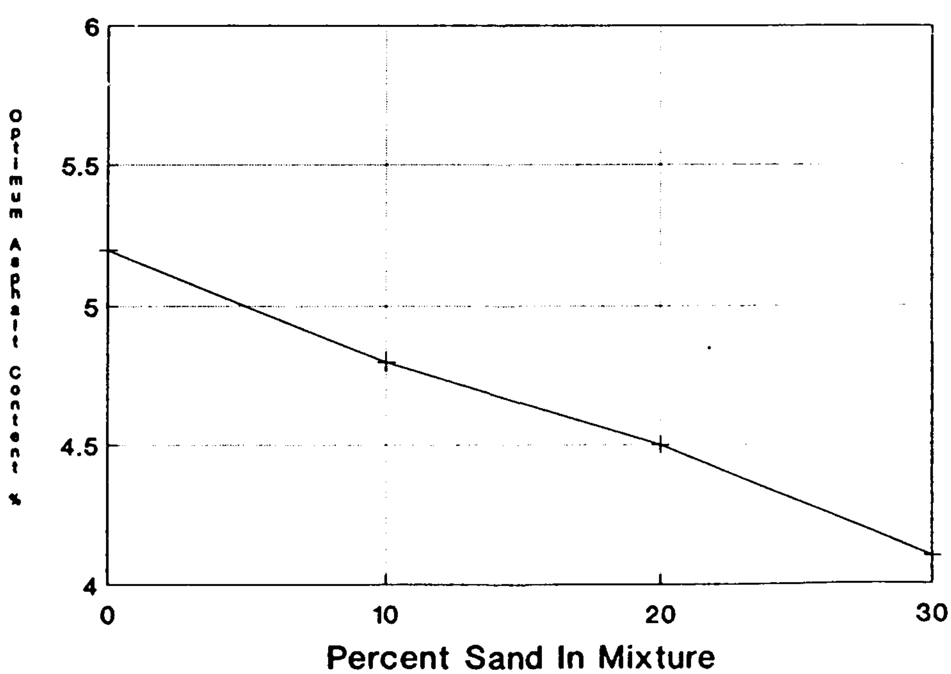


Figure 33. Optimum Asphalt Content Versus Percent Concrete Sand

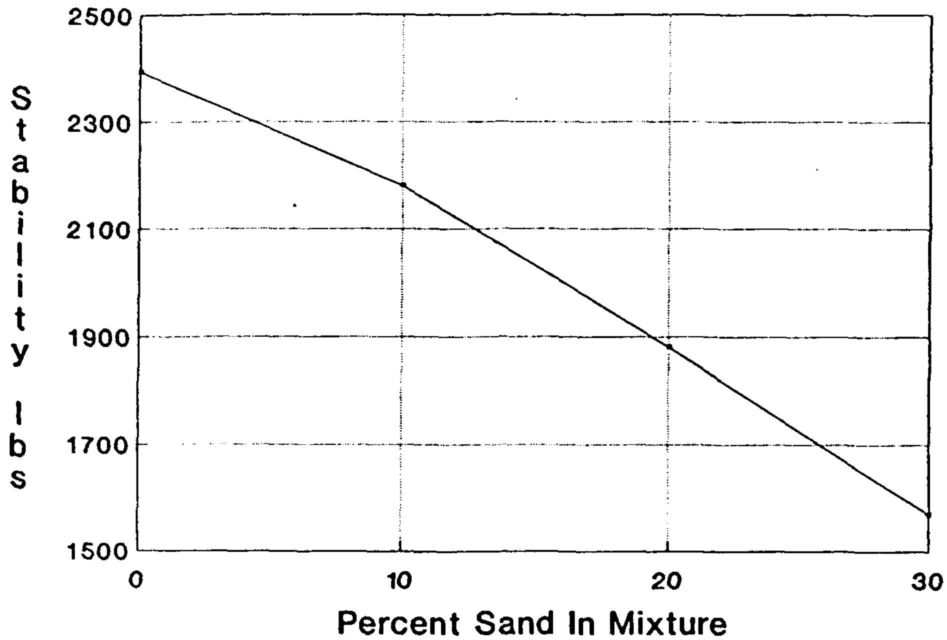


Figure 34. Marshall Stability Versus Percent Mason Sand

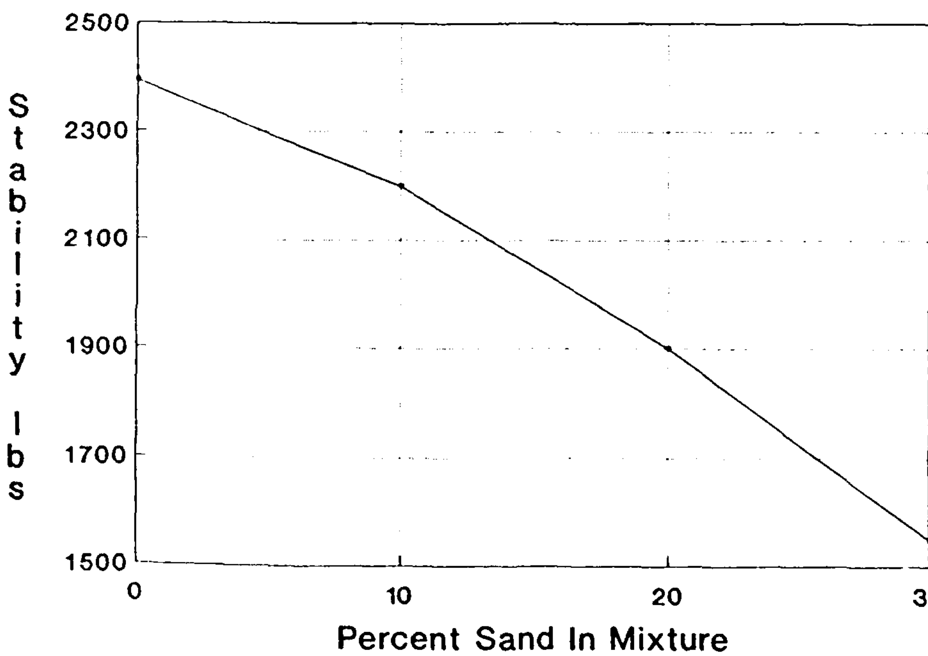


Figure 35. Marshall Stability Versus Percent Concrete Sand

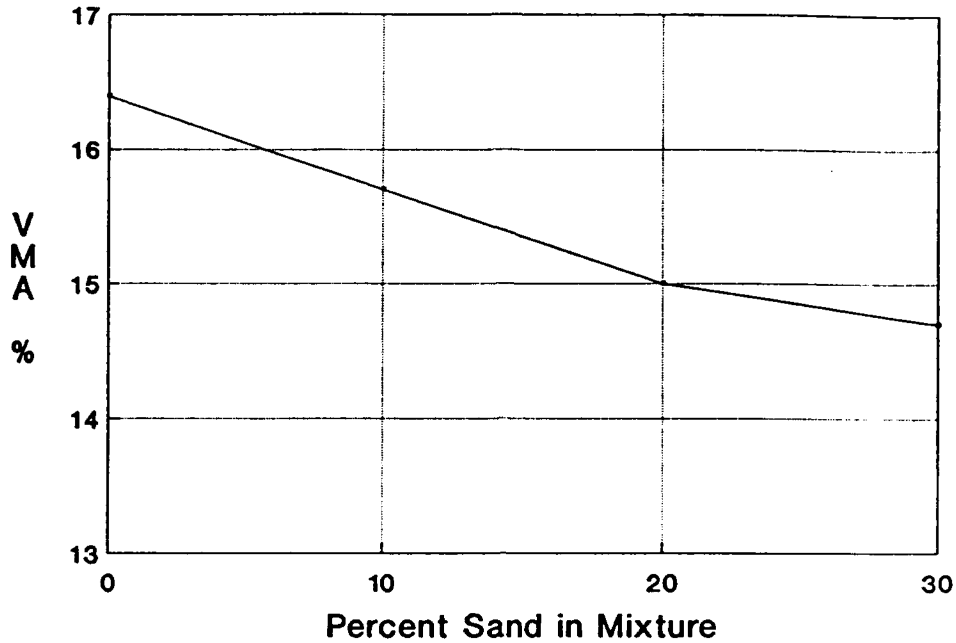


Figure 36. Voids in Mineral Aggregate Versus Percent Mason Sand

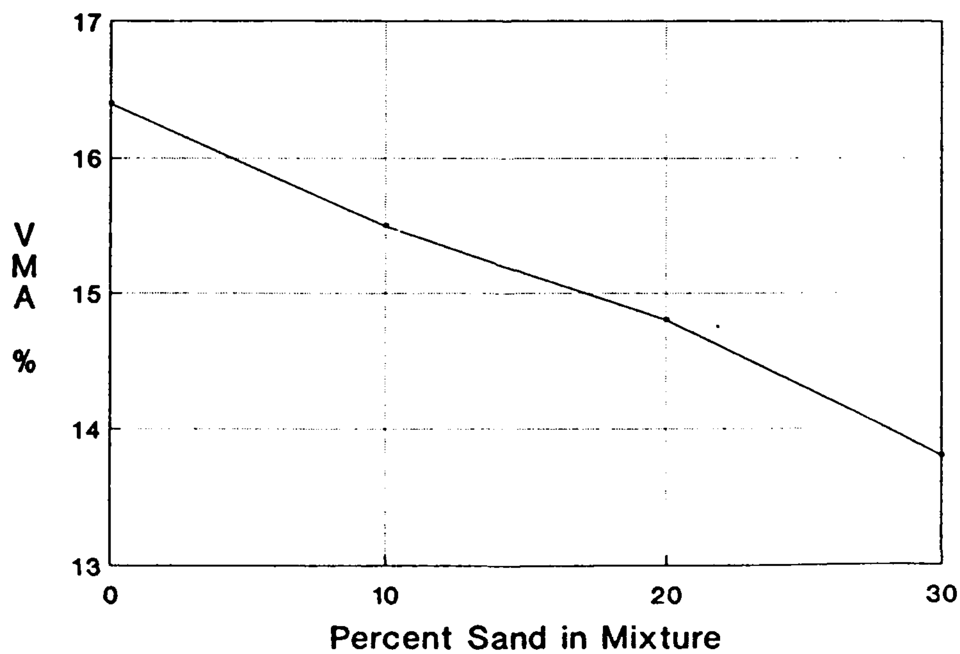


Figure 37. Voids in Mineral Aggregate Versus Percent Concrete Sand

CHAPTER V

PHASE II - PRESENTATION AND ANALYSIS OF DATA

This chapter presents and discusses the results of the laboratory testing involved in Phase II of this laboratory study. Phase II testing was developed to evaluate the effects of natural sands on asphalt concrete mixtures using state-of-the-art testing equipment. Forty asphalt concrete specimens were produced for each aggregate blend at the optimum asphalt content determined in Phase I. Each specimen was compacted with the GTM. The Marshall stability, flow and voids properties were determined for each aggregate blend. The indirect tensile, resilient modulus, and unconfined creep-rebound tests were also conducted to determine the strength characteristics of the various mixtures.

The purpose of this study was to determine the effect of natural sands on asphalt mixtures and to determine allowable limits for the natural sand content. The general approach used to analyze the test results involved a direct comparison of test values and a graphical analysis. The scope of this laboratory study allowed a direct comparison of test values because the main variable was the amount or percentage of natural sand in the mixture. Since the number of variables was limited, the comparison of these results for each test procedure was considered to be an excellent means of analyzing these mixtures.

Graphical analyses were conducted for practically all test results for this study. A large number of graphs were produced to allow a visual interpretation of the data. Graphical analyses generally demonstrate trends and tendencies and exhibit test variable relationships. The graphs produced in this study supported the expected findings and helped define certain relationships and trends.

Marshall Mix Properties

Phase II of the natural sand laboratory evaluation required that the standard Marshall mix properties be determined at the optimum asphalt content so these test values could be analyzed with the more modern, sophisticated test procedures. A summary of the Marshall mix properties for Phase II is presented in Table 8. The test results presented for the mix properties, unit weight, voids total mix, voids in mineral aggregate, voids filled with asphalt, and the gyratory elasto-plastic index (GEPI) are an average of 40 specimens. The stability and flow test results are an average of three to nine specimens.

The optimum asphalt contents that were selected in Phase I were based on mixtures having 4 percent total voids. The percent voids total mix for the specimens produced in Phase II varied slightly from the target value. The average percent voids total mix maximum variance from the target value was 0.3 percent for the S-0 aggregate blend. The remaining average values had less than an 0.2 percent variance. These variances in percent voids total mix are not considered to be significant and should not have an effect on the test results.

TABLE 8

SUMMARY OF MIX PROPERTIES AT OPTIMUM ASPHALT CONTENT

Aggregate Blend	Optimum Asphalt Content (percent)	Unit Weight (pcf)	Voids Total Mix (percent)	Voids in Mineral Aggregate (percent)	Voids Filled With Asphalt (percent)	Stability (lbs)	Flow (0.01 in)	Gyratory Elasto-Plastic Index
S-0	5.2	154.6	4.3	16.7	74.4	2393	10	1.20
S-1M	4.9	155.3	3.8	15.6	75.6	2386	9	1.40
S-2M	4.6	154.8	3.9	14.9	73.8	1817	9	1.45
S-3M	4.5	154.0	4.1	14.8	72.3	1630	9	1.50
S-1C	4.8	155.0	4.1	15.6	73.6	2166	8	1.35
S-2C	4.5	155.3	3.8	14.6	73.9	1986	7	1.40
S-3C	4.1	154.7	4.2	14.1	70.1	1581	7	1.45

The unit weight or density values did not vary significantly as the percentage of natural sand increased. The unit weight of all the crushed limestone mixture (S-0) was 154.6 pounds per cubic foot (pcf). The unit weight values for the mixtures containing natural sand did not vary significantly from the S-0 blend. The maximum unit weight value was 155.3 pcf for the S-1M and S-2C blends and the minimum unit weight value was 154.0 pcf for the S-3M blend. The difference in unit weight values from the S-0 blend is less than 1 pcf and was considered to be insignificant.

The voids filled with asphalt values indicated a general trend that these values decreased as the percentage of natural sand increased. The voids filled values for the S-0 blend was 74.4 percent. The test results showed a small variance at 10 percent natural sand, but a larger, more significant variance at 20 and 30 percent natural sand. The voids filled with asphalt value was 75.6 percent for the S-1M blend and 73.6 percent for the S-1C blend. The voids filled value decreased to 72.3 percent for the S-3M blend and 70.1 per-cent for the S-3C blend.

The voids in mineral aggregate (VMA) test results also decreased as the percentage of natural sand increased. The VMA value for the S-0 blend was 16.7 percent. The asphalt mixtures containing natural sand progressively decreased from this value. The average value was 15.6 percent for 10 percent natural sand, 14.8 percent for 20 percent natural sand, and 14.4 percent for 30 percent natural sand. Figures 38-39 show the VMA values versus percent natural sand in mixture. This relationship of decreasing VMA values with increasing percentages of natural sand is supported in both Phases I and II. This

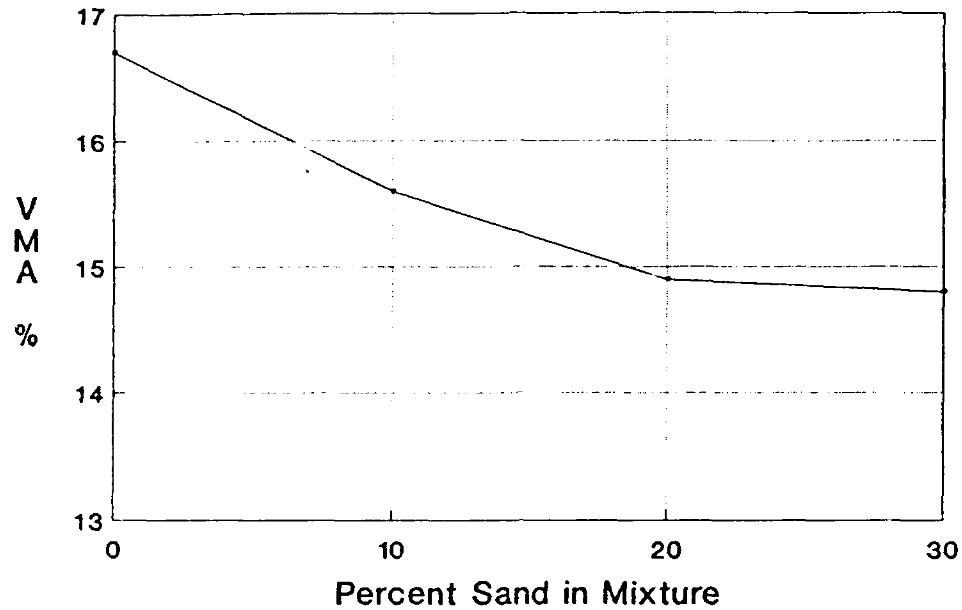


Figure 38. VMA Versus Percent Mason Sand

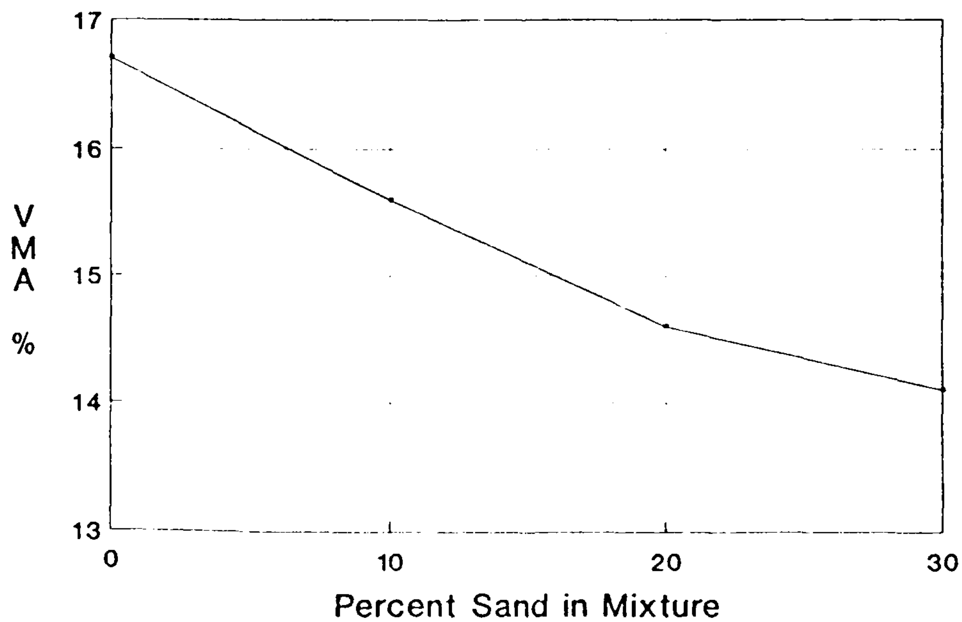


Figure 39. VMA Versus Percent Concrete Sand

reduction in VMA values indicated the potential for less stable asphalt mixtures and unsatisfactory field performance (15).

The Marshall stability test results indicate there is a direct relationship between stability and the percentage of natural sand. As the percentage of natural sand increases in an asphalt concrete mixture, the stability or resistance to deformation decreases significantly. The stability values for each aggregate blend are listed in Table 9. The stability value for the crushed aggregate mixture (S-0) had an average stability of 2393 lbs. This value is well above the 1800 lbs minimum requirement for heavy duty pavements. The decrease in stability values was minor for 10 percent natural sand, approximately 4.9 percent. The decrease was more pronounced at the 20 and 30 percent natural sand contents, 20.5 percent and 32.9 percent, respectively. At the 30 percent level of natural sand, the stability values had decreased to approximately 1600 lbs which is below the minimum requirement and not acceptable for heavy duty pavements. Table 10 lists the summary of Marshall stability values and Figures 40-41 show these values versus the percent sand in mixture.

The Marshall flow values did not indicate a significant relationship between flow values and percent natural sand. A larger effect on flow values was caused by the type of natural sand instead of percentage of sand. The mason sand had little effect on the flow of the mixtures; all mixtures had a flow of 9. The concrete sand caused a larger change; a flow value of 7 at 30 percent natural sand.

TABLE 9

MARSHALL STABILITY AND FLOW RESULTS

Aggregate Blend	Marshall Stability (lbs)	Flow (0.01 in)
S-0	2133	10
	2183	10
	2218	10
	2756	9
	2617	10
	2450	10
S-1M	2080	9
	2288	7
	2664	7
	2354	8
	2432	8
	2496	8
S-2M	1924	8
	1832	9
	1742	9
	1832	9
	1786	8
	1786	9

TABLE 9 (continued)

Aggregate Blend	Marshall Stability (lbs)	Flow (0.01 in)
S-3M	1483	9
	1578	8
	1526	9
	1768	10
	1786	10
	1638	10
S-1C	2098	8
	1950	8
	2270	9
	2054	8
	2508	8
	2184	8
	1936	8
	2328	8
S-2C	1976	7
	1964	7
	1986	7

TABLE 9 (continued)

Aggregate Blend	Marshall Stability (lbs)	Flow (0.01 in)
S-3C	1526	7
	1578	7
	1392	7
	1924	7
	1860	7
	1578	7
	1482	7
	1508	7
	1378	7

TABLE 10

SUMMARY OF MARSHALL STABILITY VALUES

Percent Natural Sand	Type of Sand	Marshall Stability (lbs)	Percent Decrease
0	Crushed	2393	--
10	Mason	2386	0.3
	Concrete	2166	9.5
	Average for 10% Sand	2276	4.9
20	Mason	1817	24.1
	Concrete	1986	17.0
	Average for 20% Sand	1902	20.5
30	Mason	1630	31.9
	Concrete	1581	33.9
	Average for 30% Sand	1606	32.9

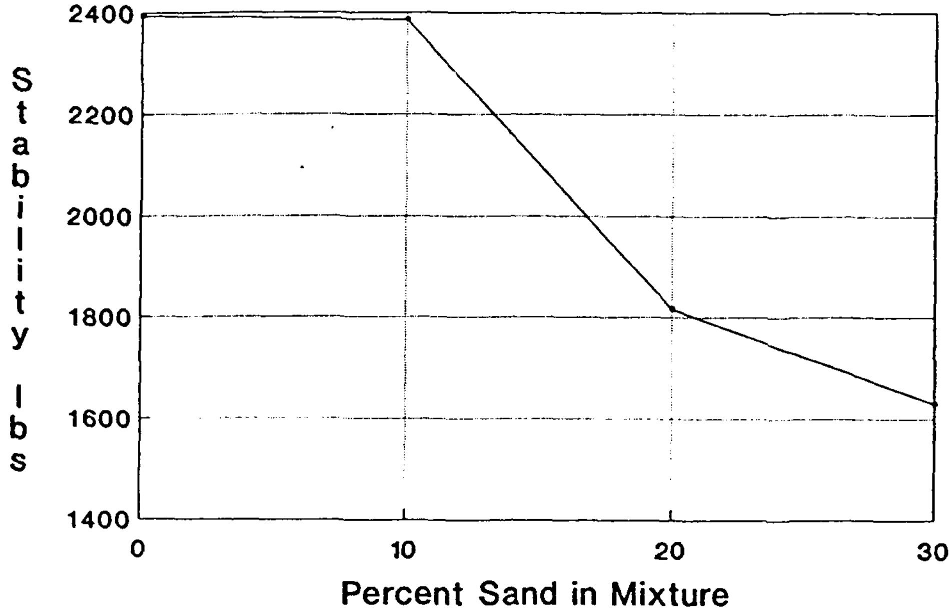


Figure 40. Marshall Stability Versus Percent Mason Sand

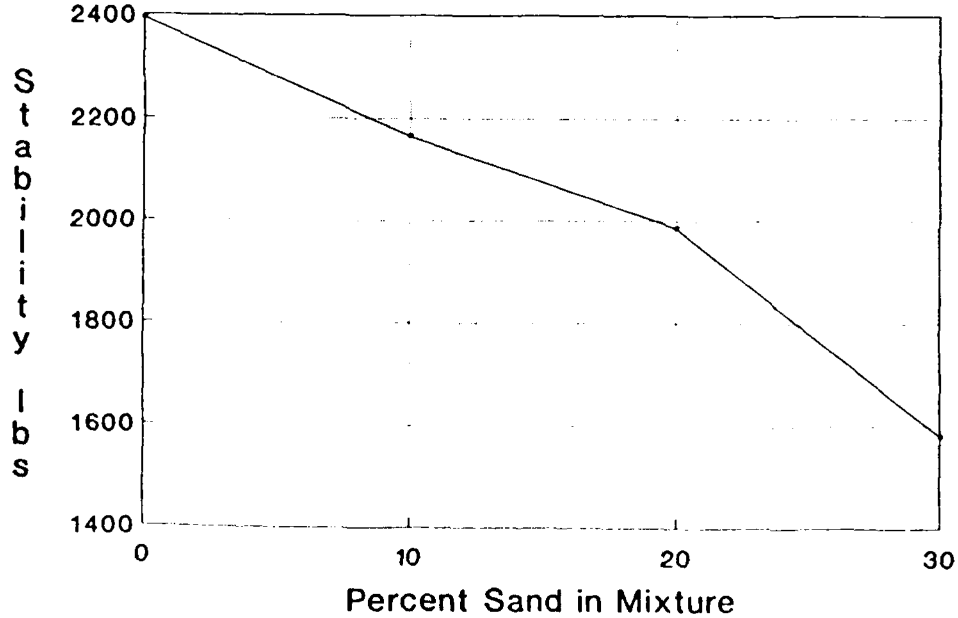


Figure 41. Marshall Stability Versus Percent Concrete Sand

Indirect Tensile

The indirect tensile test was conducted to determine the fundamental tensile strength properties of the asphalt concrete mixtures. This test was conducted on a minimum cf three specimens for each of the seven aggregate blends. The indirect tensile test was conducted at two test temperatures, 77^0F and 104^0F. These test temperatures were chosen because most pavement deformation occurs at higher temperatures. The results of the indirect tensile test are presented in Table 11.

Tensile strength values are usually dependent on the type of binder or asphalt cement material and the temperature of the testing. The test results of this study indicate that the test temperature had a significant effect on the tensile strength values. The tensile strength values at 77^0F are approximately three times greater than the tensile strength values at 104^0F. A summary of tensile strength values at 77^0F and 104^0F are presented in Tables 12-13.

The tensile strength values were also affected by the percentage of natural sand in the mixture. At 77^0F, the tensile strength of the all crushed limestone mixture (S-0) was 147.0 psi. The tensile strength values for the mixtures containing natural sand decreased as the percentage of natural sand increased. The average tensile strength value was 125.7 psi for 10 percent natural sand, 118.7 psi for 20 percent natural sand, and 116.3 psi for 30 percent natural sand. The reduction in tensile strength at 30 percent natural sand was approximately 20.9 percent. The actual tensile strength decreased for the mason s.nd specimens was 28.9 percent.

Figure 42 shows the indirect tensile strength values at 77^{0}F versus the percent natural sand in mixture.

The indirect tensile strength values at 104^{0}F were also affected by an increase in natural sand materials. The indirect tensile values decreased significantly as the percentage of natural sand increased. The indirect tensile strength for the S-0 blend was 50.1 psi. The average tensile strength value was 42.9 psi for 10 percent natural sand, 41.0 psi for 20 percent natural sand, and 37.9 psi for 30 percent natural sand. The decrease in tensile strength at the 30 percent natural sand content was 24.4 percent. Figure 43 shows the indirect tensile strength values at 104^{0}F versus the percent natural sand in the mixture.

Resilient Modulus

The resilient modulus test was conducted to evaluate the relative quality of the asphalt concrete mixtures. This test was conducted on a minimum of three specimens for each of the seven aggregate blends. Since this test was considered to be a nondestructive test, duplicate tests were conducted on each specimen. The resilient modulus test was also conducted at two test temperatures, 77^{0}F and 104^{0}F. The results of the resilient modulus test are presented in Table 14.

The resilient modulus value of an asphalt concrete mixture is generally dependent on the type of asphalt cement, aggregate gradation, and the shape and texture of the aggregate. Since this laboratory study used the same asphalt cement and primarily the same aggregate gradation, the variation in aggregate shape and texture would be analyzed.

TABLE 11

INDIRECT TENSILE TEST RESULTS

Aggregate Blend	Temperature (degrees F)	Thickness (inches)	Vertical Load (pounds)	Tensile Strength (psi)
S-0	77	2.514	2317.2	146.7
	77	2.515	2432.8	154.0
	77	2.524	2204.3	139.0
	77	2.469	2379.0	153.4
	77	2.504	2231.2	141.8
	104	2.531	811.8	51.1
	104	2.500	814.5	51.9
	104	2.492	787.6	50.3
	104	2.505	752.7	47.8
	104	2.502	776.9	49.4
S-1M	77	2.462	2024.2	130.9
	77	2.466	1922.1	124.1
	77	2.507	1954.3	124.1
	77	2.503	1908.6	121.4
	77	2.477	2013.4	129.4

TABLE 11 (continued)

Aggregate Blend	Temperature (degrees F)	Thickness (inches)	Vertical Load (pounds)	Tensile Strength (psi)
S-1M	104	2.449	750.0	48.7
	104	2.480	704.3	45.2
	104	2.495	728.5	46.5
	104	2.483	701.6	45.0
	104	2.479	707.0	45.4
S-2M	77	2.503	1798.4	114.4
	77	2.493	1828.0	116.7
	77	2.504	1743.6	110.8
	77	2.494	1771.3	113.0
	77	2.503	1710.6	108.8
	104	2.481	626.3	40.2
	104	2.503	604.8	38.5
	104	2.510	611.7	38.8
	104	2.487	729.8	46.7
	104	2.493	681.9	43.5
S-3M	77	2.484	1619.4	103.8
	77	2.511	1612.9	102.2
	77	2.497	1618.3	103.2
	77	2.503	1707.5	108.6
	77	2.504	1646.2	104.6

TABLE 11 (continued)

Aggregate Blend	Temperature (degrees F)	Thickness (inches)	Vertical Load (pounds)	Tensile Strength (psi)
S-3M	104	2.501	611.8	38.9
	104	2.521	622.6	39.3
	104	2.489	557.0	35.6
	104	2.498	548.4	34.9
	104	2.497	665.6	42.4
S-1C	77	2.490	1994.6	127.5
	77	2.492	2010.8	128.4
	77	2.484	2008.6	128.7
	77	2.492	2034.4	129.9
	77	2.509	1773.1	112.5
	104	2.496	599.5	38.2
	104	2.498	588.7	37.5
	104	2.481	646.2	41.5
	104	2.479	623.7	40.0
	104	2.453	625.8	40.6
S-2C	77	2.491	1969.9	125.9
	77	2.443	1872.0	122.0
	77	2.453	1941.9	126.0

TABLE 11 (continued)

Aggregate Blend	Temperature (degrees F)	Thickness (inches)	Vertical Load (pounds)	Tensile Strength (psi)
S-2C	104	2.495	619.4	39.9
	104	2.458	641.9	41.2
	104	2.488	673.4	39.6
	104	2.479	608.5	41.2
S-3C	77	2.501	2167.0	137.9
	77	2.493	2096.8	133.9
	77	2.496	1955.9	124.7
	77	2.518	1855.9	117.3
	77	2.483	1973.1	126.5
	104	2.500	673.4	42.9
	104	2.503	608.5	38.7
	104	2.495	554.8	35.4
	104	2.476	591.4	38.0
	104	2.481	511.8	32.8

TABLE 12

SUMMARY OF INDIRECT TENSILE TEST AT 77°F

Percent Natural Sand	Type of Sand	Tensile Strength (psi)	Percent Decrease
0	Crushed	147.0	--
10	Mason	126.0	14.3
	Concrete	125.4	14.7
	Average for 10% Sand	125.7	14.5
20	Mason	112.7	23.3
	Concrete	124.6	15.2
	Average for 20% Sand	118.7	19.3
30	Mason	104.5	28.9
	Concrete	128.1	12.9
	Average for 30% Sand	116.3	20.9

TABLE 13

SUMMARY OF INDIRECT TENSILE TEST AT 104^{0}F

Percent Natural Sand	Type of Sand	Tensile Strength (psi)	Percent Decrease
0	Crushed	50.1	--
10	Mason	46.2	7.8
	Concrete	39.6	21.0
	Average for 10% Sand	42.9	14.4
20	Mason	41.5	17.2
	Concrete	40.5	19.2
	Average for 20% Sand	41.0	18.2
30	Mason	38.2	23.8
	Concrete	37.6	25.0
	Average for 30% Sand	37.9	24.4

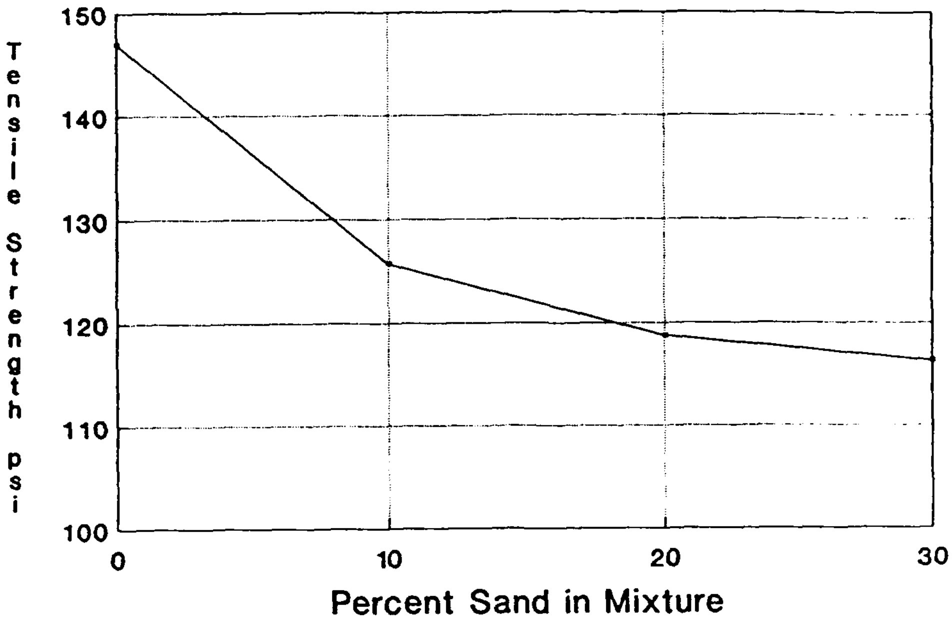


Figure 42. Indirect Tensile Strength Values at 77°F

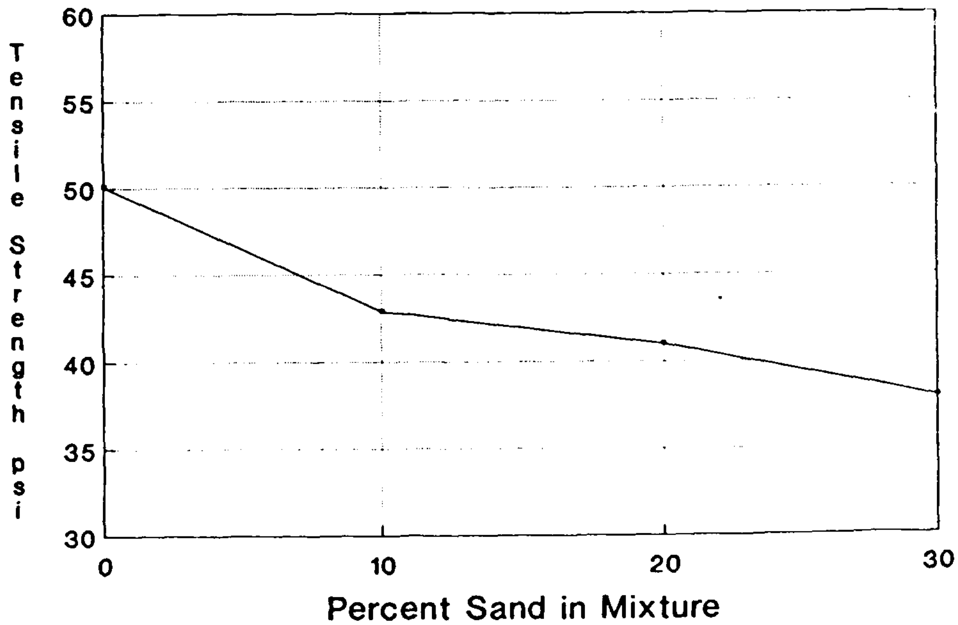


Figure 43. Indirect Tensile Strength Values at 104°F

However, the results from the resilient modulus tests were inconsistent and showed no conclusive trends.

The inconsistency of the data was very evident when duplicate test values from the same specimen were evaluated. Two-thirds of the specimens tested had results that varied from the initial test value by more than $\pm$ 20 percent. The vast majority of the second test values had increased when compared to the initial test value. The variation in test values ranged from a 50 percent decrease to a 200 percent increase. Based on this significant variation in test results, only the initial resilient modulus values were analyzed. Two initial test values were also eliminated because these values were approximately five times greater than the other two specimens at the same asphalt content and gradation. These test values were approximately two million psi, not typical values for an asphalt concrete mixture at 77^0F.

The resilient modulus values that were analyzed indicated that the test temperature and the amount of natural sand did effect the resilient modulus values. The resilient modulus values at 77^0F were three to five times greater than the resilient modulus values at 104^0F. The resilient modulus values also decreased as the percentage of natural sand increased, but the values were inconsistent. A summary of the resilient modulus values at 77^0F and 104^0F are presented in Table 15.

The resilient modulus values at 77^0F indicated the type of natural sand had some effect on the resilient modulus value. The resilient modulus value for the crushed limestone mixture (S-0) was 589,192 psi. The resilient modulus value was 547,194 psi for 10 percent mason sand, 465,744 psi for 20 percent mason sand, and 390,828 psi for 30 percent mason sand. The resilient modulus value was 492,214 psi for 10 concrete sand, 423,814 psi for 20 percent concrete sand, and 579,898 psi for 30 percent concrete sand. The various amounts of natural sand did not develop a true relationship for the resilient modulus value at 77°F. Figure 44 presents the resilient modulus values at 77°F versus the percent sand in mixture.

The resilient modulus values at 104°F also indicated an inconsistent relationship between the resilient modulus value and the percentage of natural sand in the asphalt concrete mixture. The resilient modulus value for the S-0 blend was 190,354 psi. The resilient modulus value was 164,722 psi for 10 percent mason sand, 199,522 psi for 20 percent mason sand, and 147,414 psi for 30 percent mason sand. The resilient modulus value was 99,412 psi for 10 percent concrete sand, 126,833 psi for 20 percent concrete sand, and 140,431 psi for 30 percent concrete sand. These resilient modulus values are varied enough to be considered inconsistent. Figure 45 presents the resilient modulus values at 104°F versus the percent natural sand in mixture.

TABLE 14

RESILIENT MODULUS TEST RESULTS

Aggregate Blend	Temperature (degrees)	Thickness (inches)	Rotation (degrees)	Vertical Load (pounds)	Horizontal Deformation (inches 10^{-4})	Resilient Modulus (psi)
S-0	77	2.524	0	58.064	0.266	535,436
	77	2.524	90	58.064	0.160	892,383
	77	2.469	0	59.139	0.266	557,500
	77	2.469	90	58.064	0.107	1,368,410
	77	2.504	0	58.064	0.213	674,641
	77	2.504	90	58.064	0.213	674,641
	104	2.492	0	17.024	0.160	267,808
	104	2.492	90	18.279	0.320	142,273
	104	2.505	0	17.204	0.320	133,209
	104	2.505	90	17.204	0.266	159,851
	104	2.502	0	18.279	0.266	170,045
	104	2.502	90	17.204	0.213	200,053

TABLE 14 (continued)

Aggregate Blend	Temperature (degrees)	Thickness (inches)	Rotation (degrees)	Vertical Load (pounds)	Horizontal Deformation (inches 10^{-4})	Resilient Modulus (psi)
S-1M	77	2.507	0	36.559	0.159	567,194
	77	2.507	90	37.634	0.213	437,907
	77	2.503	0	36.559	0.478	189,367
	77	2.503	90	37.634	0.159	584,809
	77	2.477	0	37.634	0.106	886,422
	77	2.477	90	37.634	0.213	443,211
	104	2.495	0	16.129	0.213	187,776
	104	2.495	90	15.053	0.267	140,206
	104	2.483	0	15.053	0.320	117,403
	104	2.483	90	16.129	0.213	188,683
	104	2.479	0	16.129	0.213	188,987
	104	2.479	90	15.053	0.213	176,388

TABLE 14 (continued)

Aggregate Blend	Temperature (degrees)	Thickness (inches)	Rotation (degrees)	Vertical Load (pounds)	Horizontal Deformation (inches 10^{-4})	Resilient Modulus (psi)
S-2M	77	2.504	0	43.010	0.214	497,338
	77	2.504	90	43.010	0.214	497,337
	77	2.494	0	44.086	0.214	511,815
	77	2.494	90	43.010	0.321	332,888
	77	2.503	0	41.935	0.268	388,078
	77	2.503	90	43.010	0.321	331,691
	104	2.510	0	12.903	0.161	198,459
	104	2.510	90	12.903	0.161	198,460
	104	2.487	0	12.903	0.161	200,295
	104	2.487	90	13.978	0.482	72,329
	104	2.493	0	12.903	0.161	199,813
	104	2.493	90	13.978	0.214	162,348

TABLE 14 (continued)

Aggregate Blend	Temperature (degrees)	Thickness (inches)	Rotation (degrees)	Vertical Load (pounds)	Horizontal Deformation (inches 10^{-4})	Resilient Modulus (psi)
S-3M	77	2.497	0	33.330	0.214	387,345
	77	2.497	90	34.408	0.214	399,840
	77	2.503	0	33.333	0.214	386,416
	77	2.503	90	33.333	0.214	515,222
	77	2.504	0	34.408	0.160	398,722
	77	2.504	90	33.333	0.214	515,016
	104	2.489	0	10.752	0.107	250,703
	104	2.489	90	9.677	0.053	451,267
	104	2.498	0	10.752	0.214	124,900
	104	2.498	90	10.752	0.160	166,533
	104	2.497	0	8.602	0.321	66,640
	104	2.497	90	10.752	0.160	166,600

TABLE 14 (continued)

Aggregate Blend	Temperature (degrees)	Thickness (inches)	Rotation (degrees)	Vertical Load (pounds)	Horizontal Deformation (inches 10^{-4})	Resilient Modulus (psi)
S-1C	77	2.484	0	48.387	0.264	457,247
	77	2.484	90	48.387	0.158	762,078
	77	2.492	0	47.311	0.158	742,750
	77	2.492	90	48.387	0.211	569,723
	77	2.509	0	47.311	0.423	276,644
	77	2.509	90	46.236	0.264	432,570
	104	2.481	0	11.828	0.317	93,255
	104	2.481	90	12.903	0.106	305,199
	104	2.453	0	10.752	0.211	128,618
	104	2.453	90	10.752	0.106	257,236
	104	2.479	0	9.677	0.317	76,362
	104	2.479	90	12.903	0.264	122,178

TABLE 14 (continued)

Aggregate Blend	Temperature (degrees)	Thickness (inches)	Rotation (degrees)	Vertical Load (pounds)	Horizontal Deformation (inches 10^{-4})	Resilient Modulus (psi)
S-2C	77	2.479	0	39.784	0.264	377,313
	77	2.479	90	39.784	0.211	471,642
	77	2.486	0	39.784	0.211	470,314
	77	2.486	90	39.784	0.105	940 628
	77	2.453	0	39.784	0.053	1,906,560
	77	2.453	90	40.861	0.264	391,619
	104	2.488	0	12.903	0.274	60,947
	104	2.488	90	13.978	0.211	165,113
	104	2.466	0	13.978	0.211	166,586
	104	2.466	90	11.828	0.369	80,546
	104	2.479	0	12.903	0.211	152,965
	104	2.479	90	12.903	0.105	305,330

TABLE 14 (continued)

Aggregate Blend	Temperature (degrees)	Thickness (inches)	Rotation (degrees)	Vertical Load (pounds)	Horizontal Deformation (inches 10^{-4})	Resilient Modulus (psi)
S-3C	77	2.496	0	41.935	0.210	495,573
	77	2.496	90	43.010	0.210	508,280
	77	2.518	0	40.860	0.053	1,914,590
	77	2.518	90	41.935	0.105	982,486
	77	2.483	0	41.935	0.158	664,223
	77	2.483	90	40.860	0.158	647,192
	104	2.495	0	12.903	0.263	121,972
	104	2.495	90	12 903	0.263	121,972
	104	2.476	0	13.978	0.210	166,438
	104	2.476	90	15.053	0.210	179,241
	104	2.481	0	13.978	0.263	132,882
	104	2.481	90	13.978	0.210	166,102

TABLE 15

SUMMARY OF RESILIENT MODULUS TEST RESULTS

Percent Natural Sand	Type of Sand	Resilient Modulus 77^0F (psi)	Resilient Modulus 104^0F (psi)
0	Crushed	589,192	190,354
10	Mason	547,194	164,722
	Concrete	492,214	99,412
20	Mason	465,744	199,522
	Concrete	423,814	126,833
30	Mason	390,828	147,414
	Concrete	579,898	140,431

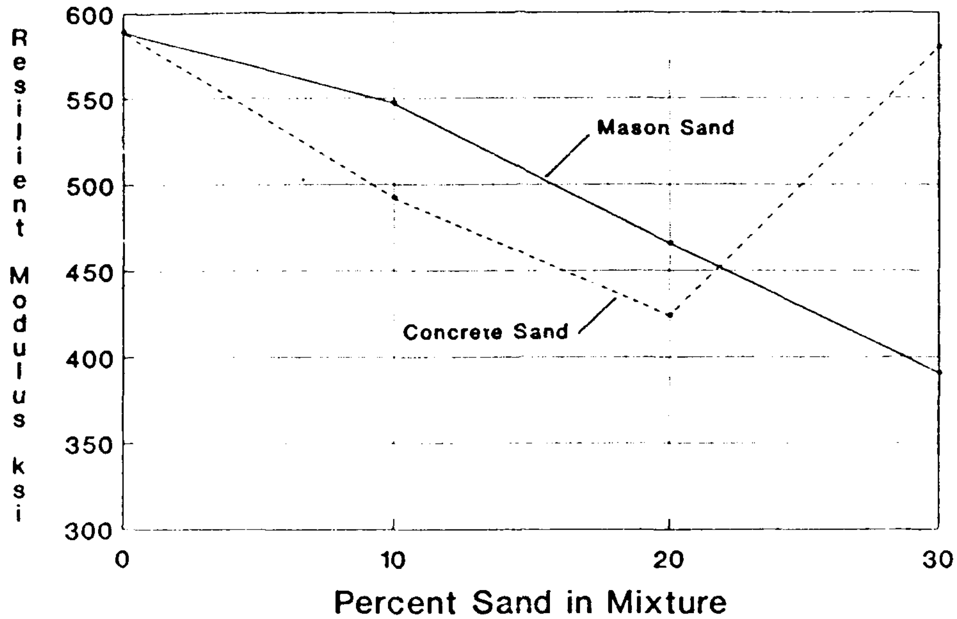


Figure 44. Resilient Modulus Values at 77°F

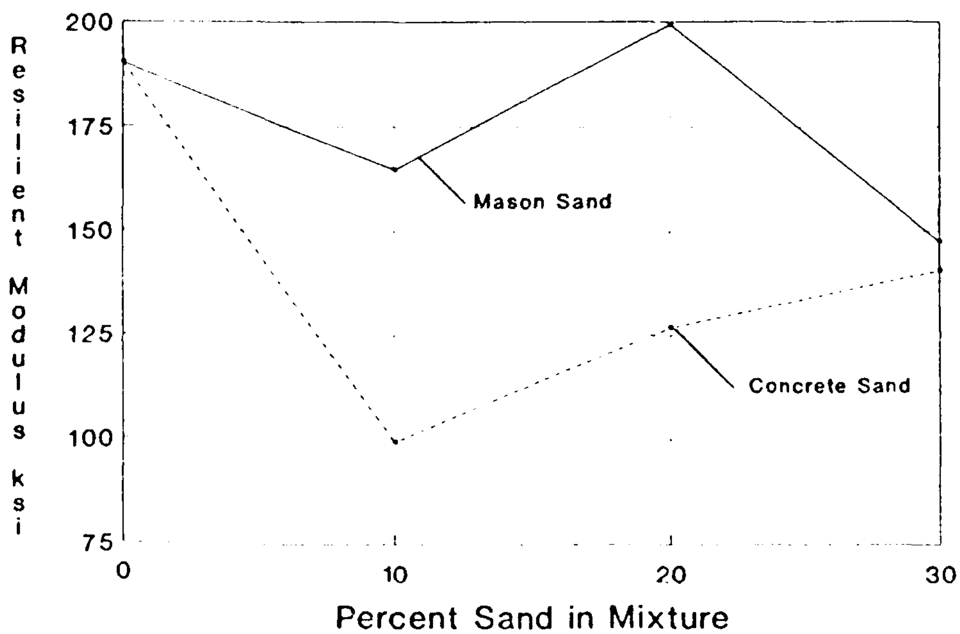


Figure 45. Resilient Modulus Values at 104°F

Unconfined Creep-Rebound

The unconfined creep-rebound test was conducted to evaluate the ability of the seven asphalt concrete mixtures to resist permanent deformation under severe loads. The creep-rebound test is one of the best laboratory procedures to determine rutting potential. The unconfined creep-rebound test was conducted at 77^0F and 104^0F and at loads that would produce a significant creep-rebound curve. The results of the unconfined creep-rebound test are presented in Table 16. Typical creep-rebound curves displaying axial deformation versus time are shown in the Appendix.

A constant vertical load was desired to test all aggregate blends for each test temperature. The vertical load was selected to produce significant deformation in the stronger mixtures and not to overload the weaker mixtures. The initial vertical load was 40 psi for 77^0F tests and 15 psi for 104^0F tests. The 40 psi load worked satisfactorily until the 30 percent natural sand mixtures were tested. At the 30 percent natural sand content, the mixtures failed and the vertical load was decreased to 30 psi. A 15 psi vertical load was used to test the 0, 10, and 20 per-cent specimens at 104^0F. This vertical load was decreased to 10 psi for 30 percent mason sand and 20 and 30 percent concrete sand mixtures because these asphalt concrete mixtures failed at the higher initial load.

The results of the unconfined creep-rebound test were used to evaluate the seven asphalt concrete mixtures. The amount of axial deformation during the loading or creep phase indicated the ability of the mixture to resist deformation. Small axial deformations indicate stable mixtures with good resistance to deformation. The calculated creep modulus indicated the stiffness of the asphalt mixtures. High creep modulus values are desired to decrease rutting potential. The percent rebound or recovered deformation indicated the ability of the mixture to recover the traffic-induced deformation. High percent rebound values indicate that permanent deformation will be minimum.

The amount of natural sand affected the test results of the unconfined creep-rebound test at both test temperatures. A relationship between the percentage of natural sand and the amount of axial deformation, creep modulus, and percent rebound was determined. The overall tendency was that the asphalt concrete mixtures weakened or increased in rutting potential as the natural sand content increased. A summary of the unconfined creep-rebound test values at 77^0F and 104^0F are presented in Tables 17-18.

The creep-rebound values at 77^0F indicated a significant relationship between the natural sand content and the creep-rebound properties. The axial deformation of the crushed limestone mixture (S-0) was 0.0058 inches. The axial deformation for the mixtures containing natural sand increased as the percentage of natural sand increased. The average axial deformation was 0.0089 inches for 10 percent natural sand, 0.0106 inches for 20 percent natural sand, and 0.0114 inches for 30 percent natural sand. The increase in axial deformation was 53.4 percent at 10 percent natural sand, 82.8 percent at 20 percent natural sand, and 96.6 percent at 30 percent natural sand. Figure 46 displays the axial deformation values at 77^0F versus the percent natural sand in mixture.

The permanent deformation values also increased as the natural sand content increased. The permanent deformation value for the S-0 blend was 0.0039 inches. The average permanent deformation was 0.0069 inches, a 76.9 percent increase for 10 percent natural sand. The average permanent deformation was 0.0082 inches, a 110.3 percent increase for 20 percent natural sand. The average permanent deformation was 0.0092 inches, a 136.0 percent increase for 30 percent natural sand. Figure 47 displays the permanent deformation at 77^{0}F versus the percent natural sand in mixture.

The percent rebound values decreased as the natural sand increased. The percent rebound for the crushed limestone mixture (S-0) was 33.2 percent. The average percent rebound was 27.5 percent for 10 percent natural sand, 23.3 percent for 20 percent natural sand, and 20.4 percent for 30 percent natural sand. These values indicated that less deformation was recovered as the natural sand content increased.

The creep modulus values decreased as the percentage of natural sand increased. The creep modulus values at 77^{0}F are summarized in Tables 17 and 19. The creep modulus value for the S-0 blend was 57,129 psi. The average creep modulus value was 36,899 psi for 10 percent natural sand, 31,085 psi for 20 natural sand and 22,553 psi for 30 percent natural sand. The decrease in creep modulus was significant as the natural sand content increased. The decrease in creep modulus was 35.4 percent at 10 percent natural sand, 45.6 percent at 20 percent natural sand, and 60.5 percent at 30 percent natural sand. Figure 48 displays the creep modulus values versus the percent sand in mixture.

The creep-rebound values at 104^{0}F also indicated a significant relationship between the natural sand content and the creep-rebound properties. The test results are not as consistent as the values at

77^0F, but do show the expected tendencies. Since different vertical loads were used, a direct comparison of deformations cannot be graphically analyzed. The tendencies observed in the 77^0F tests were also evident in the axial and permanent deformation values. In both creep-rebound properties, the deformation increased as the natural sand content increased.

The creep modulus values also decreased as the percentage of natural sand increased. The creep modulus values at 104^0F are summarized in Tables 18-19. The creep modulus value for the S-0 blend was 23,872 psi. The average creep modulus was 15,816 psi for 10 percent natural sand, 12,549 psi for 20 percent natural sand, and 10,216 psi for 30 percent natural sand. The decrease in creep modulus values was 33.8 percent at 10 percent natural sand, 47.5 percent for 20 percent natural sand, and 57.2 percent at 30 percent natural sand. The decrease in creep modulus at 104^0F is also significant. Figure 49 displays the creep modulus values versus the percent natural sand in mixture.

TABLE 16

UNCONFINED CREEP-REBOUND TEST RESULTS

Aggregate Blend	Temperature (degrees-F)	Vertical Load (psi)	Axial Deformation (inches)	Creep Modulus (psi)	Permanent Deformation (inches)	Rebound (percent)
S-0	77	40	0.0070	46,207	0.0049	30.0
	77	40	0.0049	66,209	0.0030	38.7
	77	40	0.0055	58,972	0.0038	30.9
	104	15	0.0053	23,035	0.0032	39.6
	104	15	0.0047	26,559	0.0032	31.9
	104	15	0.0055	22,023	0.0032	41.8
S-1M	77	40	0.0083	38,646	0.0083	36.1
	77	40	0.0098	32,956	0.0070	28.5
	104	15	0.0077	15,596	0.0051	33.7
	104	15	0.0064	18,513	0.0047	26.5
	104	15	0.0075	15,329	0.0053	29.3

TABLE 16 (continued)

Aggregate Blend	Temperature (degrees-F)	Vertical Load (psi)	Axial Deformation (inches)	Creep Modulus (psi)	Permanent Deformation (inches)	Rebound (percent)
S-2M	77	40	0.0101	31,809	0.0079	21.7
	77	40	0.0126	25,636	0.0101	19.8
	104	15	0.0079	14,617	0.0051	35.4
	104	15	-- (a)	--	--	--
	104	15	0.0105	10,796	0.0086	18.0
	104	15	-- (a)	--	--	--
S-3M	77	40	-- (a)	--	--	--
	77	30	0.0174	14,131	0.0146	16.0
	77	30	0.0094	25,729	0.0068	27.6
	77	30	0.0126	19,612	0.0105	16.6
	104	15	-- (a)	--	--	--
	104	10	0.0070	11,156	0.0053	24.2
	104	10	0.0070	11,402	0.0051	27.1
	104	10	0.0070	10,686	0.0043	38.5

(a) Sample failure-overloaded

TABLE 16 (continued)

Aggregate Blend	Temperature (degrees-F)	Vertical Load (psi)	Axial Deformation (inches)	Creep Modulus (psi)	Permanent Deformation (inches)	Rebound (percent)
S-1C	77	40	0.0073	44,222	0.0053	27.3
	77	40	0.0101	31,771	0.0083	17.8
	104	15	0.0079	14,958	0.0051	35.4
	104	15	0.0086	13,764	0.0066	23.2
	104	15	0.0068	16,734	0.0047	30.8
S-2C	77	40	0.0101	32,133	0.0079	21.7
	77	40	0.0113	28,514	0.0086	23.8
	77	40	0.0081	39,692	0.0055	32.0
	104	15	-- (a)	--	--	--
	104	10	0.0111	6,895	0.0077	30.6
	104	10	0.0047	16,631	0.0019	59.5
	104	10	0.0058	13,647	0.0032	44.8

(a) Sample failure-overloaded

TABLE 16 (continued)

Aggregate Blend	Temperature (degrees-F)	Vertical Load (psi)	Axial Deformation (inches)	Creep Modulus (psi)	Permanent Deformation (inches)	Rebound (percent)
S-3C	77	40	-- (a)	--	--	--
	77	30	0.0094	25,709	0.0081	13.8
	77	30	0.0086	28,434	0.0064	25.5
	77	30	0.0111	21,700	0.0086	22.5
	104	10	0.0092	8,630	0.0062	32.6
	104	10	0.0090	8,777	0.0058	35.5
	104	10	0.0073	10,647	0.0047	35.6

(a) Sample failure-overloaded

TABLE 17

SUMMARY OF UNCONFINED CREEP-REBOUND TEST AT 77^{0}F

Percent Natural Sand	Type of Sand	Vertical Load (psi)	Axial Deformation (inches)	Creep Modulus (psi)	Permanent Deformation (inches)	Rebound (percent)
0	Crushed	40	0.0058	57,129	0.0039	33.2
10	Mason	40	0.0091	35,801	0.0070	32.3
	Concrete	40	0.0087	37,997	0.0068	22.6
	Average for 10% Sand		0.0089	36,899	0.0069	27.5
20	Mason	40	0.0114	28,723	0.0090	20.8
	Concrete	40	0.0098	33,446	0.0073	25.8
	Average for 20% Sand		0.0106	31,085	0.0082	23.3
30	Mason	30	0.0131	19,824	0.0106	20.1
	Concrete	30	0.0097	25,281	0.0077	20.6
	Average for 30% Sand		0.0114	22,553	0.0092	20.4

TABLE 18

SUMMARY OF UNCONFINED CREEP-REBOUND TEST AT 104°F

Percent Natural Sand	Type of Sand	Vertical Load (psi)	Axial Deformation (inches)	Creep Modulus (psi)	Permanent Deformation (inches)	Rebound (percent)
0	Crushed	15	0.0052	23,872	0.0032	33.2
10	Mason	15	0.0072	16,479	0.0050	29.8
	Concrete	15	0.0077	15,152	0.0055	29.8
20	Mason	15	0.0092	12,707	0.0069	26.7
	Concrete	10	0.0072	12,391	0.0043	45.0
30	Mason	10	0.0070	11,081	0.0049	29.3
	Concrete	10	0.0085	9,351	0.0056	34.6

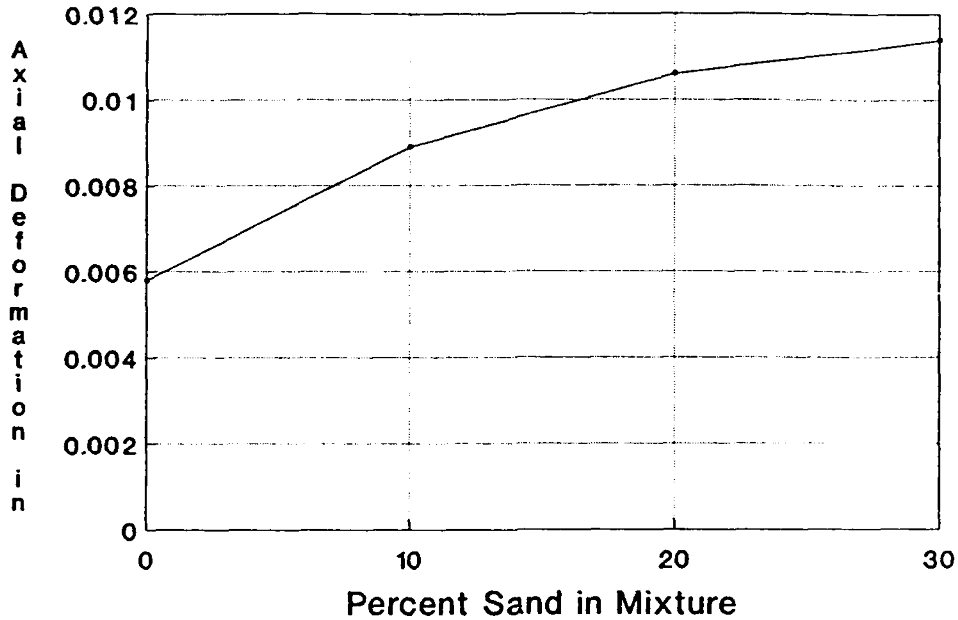


Figure 46. Axial Deformation Values at 77°F

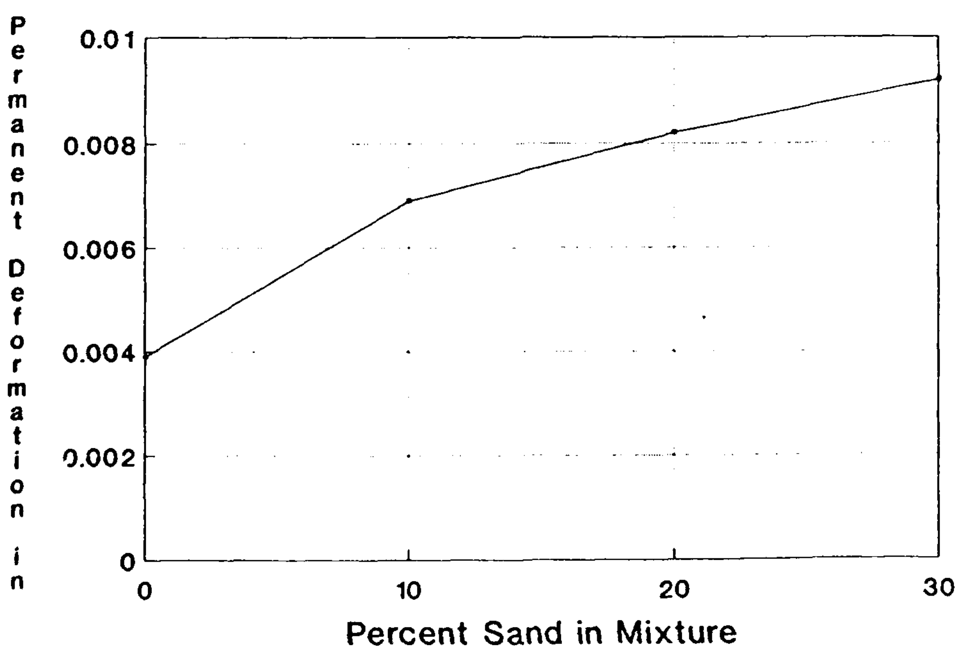


Figure 47. Permanent Deformation Values at 77°F

TABLE 19

SUMMARY OF CREEP MODULUS VALUES

Percent Natural Sand	Type of Sand	Creep Modulus (@77°F)	Decrease (percent)	Creep Modulus (@104°F)	Decrease (percent)
0	Crushed	57,129		23,872	
10	Mason	35,801	37.3	16,479	31.0
	Concrete	37,997	33.5	15,152	36.5
	Average	36,899	35.4	15,816	33.8
20	Mason	28,723	49.7	12,707	46.8
	Concrete	33,446	41.5	12,391	48.1
	Average	31,085	45.6	12,549	47.5
30	Mason	19,824	65.3	11,081	53.6
	Concrete	25,281	55.7	9,351	60.8
	Average	22,553	60.5	10,216	57.2

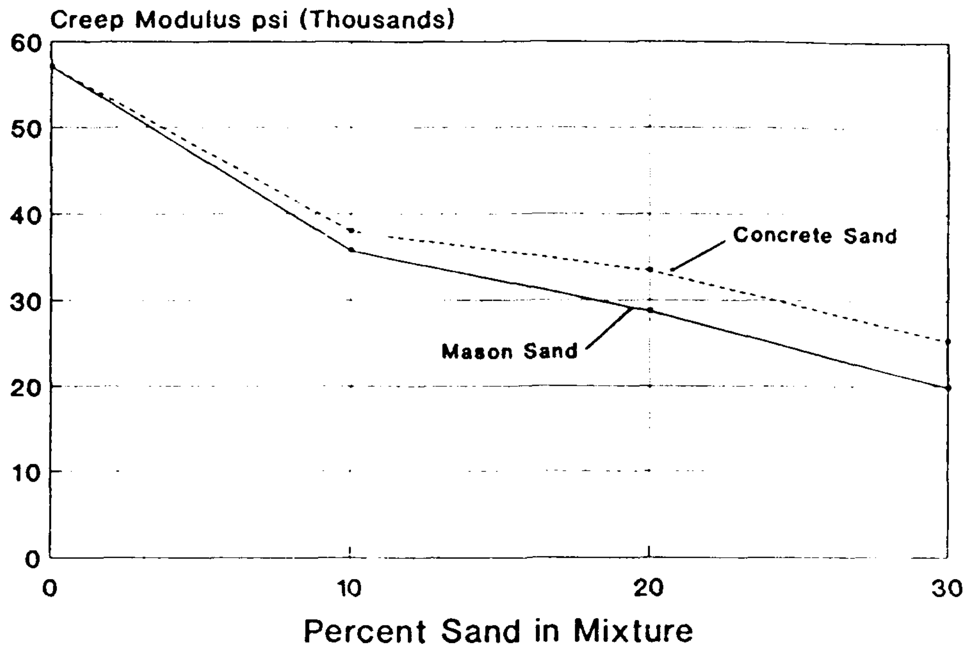


Figure 48. Creep Modulus Values at 77°F

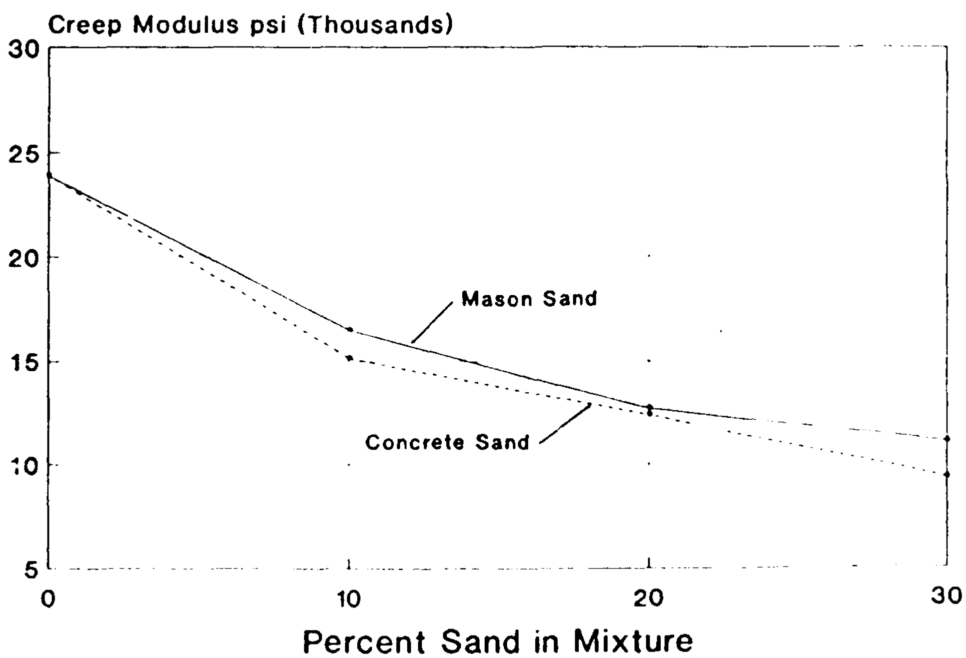


Figure 49. Creep Modulus Values at 104°F

CHAPTER VI

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Summary

This laboratory study was conducted to evaluate the effects of natural sands on the engineering properties of asphalt concrete mixtures. This research program consisted of a review of available literature and existing data, and a two-phase laboratory study on laboratory-produced specimens. Conventional and state-of-the-art testing procedures and equipment were used to determine the effects of natural sands on asphalt concrete mixtures. The objective of this research was to examine the engineering properties of the asphalt concrete mixtures and to set quantitative limits of natural sand to prevent unstable mixtures and reduce rutting potential.

The review of the literature and existing data indicated that the quality and size of the aggregate had a tremendous effect on the properties of asphalt concrete mixtures. Several laboratory research studies had been conducted comparing natural or uncrushed aggregates to crushed coarse and fine aggregates. The conclusions of these laboratory studies indicated that stability and strength properties of mixtures decreased as the percentage of uncrushed aggregates increased. These studies also indicated that replacing natural sand materials with crushed sands would increase the resistance to permanent deformation in asphalt concrete pavements.

The first phase of this laboratory study evaluated the physical properties of the materials used in this study. Aggregate gradations were computed to produce aggregate blends that were as consistent as possible. Asphalt concrete mix designs were conducted on the seven aggregate blends to select optimum asphalt contents.

The aggregate blends were produced using 0, 10, 20, and 30 percent natural sand. These blends were fabricated as close as possible to the target gradation. However, the aggregate blends for the 20 and 30 percent natural sand contents did have some variation, especially at the No. 30 sieve. A definite hump developed at the No. 30 sieve when these gradations were plotted on standard semi-log graphs and graphs with sieve sizes raised to the 0.45 power. This hump in the gradation curves indicated that asphalt mixtures with 20 and 30 percent natural sand contents were sensitive and tender.

The optimum asphalt content was determined for each aggregate blend using the Marshall mix design procedure. Several trends were evident from the mixture properties at the optimum asphalt contents. The optimum asphalt content decreased as the percentage of natural sand increased. The stability values were also effected by the percentage of natural sand; the stability values decreased as the percentage of natural sand increased. Another relationship that was observed was a decrease in voids in mineral aggregate as the percentage of natural sand increased. Each of these trends or relationships indicated that the quality and durability of the asphalt concrete mixture both decreased as the percentage of natural sand increased.

The second phase of the laboratory study evaluated the effects of natural sands on asphalt concrete mixtures using state-of-the-art testing equipment. Specimens were produced for each aggregate blend at the optimum asphalt content and evaluated with the Marshall procedure, indirect tensile test, resilient modulus test, and unconfined creep-rebound test.

The Marshall mix properties were determined so these values could be analyzed with the more modern test procedures. The test properties determined in Phase II agreed with the trends and relationships observed in Phase I. The mix properties including stability, voids filled with asphalt, and voids in mineral aggregate decreased as the percentage of natural sand increased. These Marshall properties indicated that natural sand materials lowered the strength properties and would affect the durability of the asphalt mixture by decreasing the asphalt content and void properties.

The indirect tensile test was conducted to determine the tensile strength properties of the seven asphalt concrete mixtures. The tensile strength values were effected by the percentage of natural sand and the test temperature. The relationship was evident that the amount of natural sand controlled the strength properties of the mixtures. As the natural sand content increased, the tensile strength decreased. The test temperature significantly affected the tensile strength; at 104^0F the tensile strength was three times less than the tensile strength values at 77^0F. The tensile strength was much lower at 104^0F, which indicated rutting potential would be greater at higher pavement temperatures.

The resilient modulus test was conducted to determine the relative quality of the asphalt concrete mixtures. The resilient modulus values produced in this study were very inconsistent. The ASTM procedure used to determined the resilient modulus relies heavily on measuring very small deformations. This measurement is very sensitive and produces large variations in the results. The consistency of the resilient modulus values determined in this study was not satisfactory. The unreliability of resilient modulus values has also been documented by Brown and Foo (7).

The unconfined creep-rebound test is considered one of the best laboratory procedures to determine rutting potential in asphalt concrete mixtures. This test procedure evaluated the ability of the mixtures to resist permanent deformation under severe loads. The unconfined creep-rebound values indicated that the rutting potential of asphalt concrete mixtures increased as the percentage of natural sand increased. The axial and permanent deformations were larger at higher natural sand contents. The creep modulus value decreased as the percentage of natural sand increased. The stiffness of the mixtures was much lower at 104^0F, which indicated rutting potential was greater at higher pavement temperatures.

Conclusions

Based on the results of the laboratory investigation which included the literature review and two-phase laboratory study, the following conclusions were made on the effects of natural sands on engineering properties of asphalt concrete mixtures:

1. The use of natural sand materials decreased the stability and strength characteristics of asphalt concrete mixtures.
2. Replacing natural sand materials with crushed sand materials increased the resistance to permanent deformation in asphalt concrete mixtures.
3. High natural sand contents, 20 percent and higher, caused aggregate blending problems. These natural sand contents produced gradations with high percentages of material passing the No. 30 sieve.
4. Aggregate gradations with 20 and 30 percent natural sand produced a definite hump at the No. 30 sieve when using a grading curve with the sieve sized raised to the 0.45 power.
5. Optimum asphalt content values decreased as the percentage of natural sand increased. The asphalt content required to produce a mixture at 4 percent voids total mix was much lower for a mixture with high natural sand content. Lower asphalt contents produce a less durable pavement.
6. Marshall stability values decreased as the percentage of natural sand increased. The stability values were significantly reduced at the 20 and 30 percent natural sand contents. The stability values decreased to a level that was below the minimum 1800 lbs requirement at 30 percent natural sand.
7. The voids in mineral aggregate (VMA) decreased as the percentage of natural sand increased.
8. The indirect tensile results indicated a reduction in mixture strength as the percentage of natural sand increased. The temperature of the indirect tensile test significantly effected the

tensile strength value. The higher temperature produced lower strength values. This test procedure indicated a definite trend when evaluating the natural sand content.

9. The resilient modulus test results were very inconsistent and indicated no trend. This test procedure was not a good test procedure to evaluate the effects of natural sands in asphalt concrete mixtures. The variation in test results for duplicate samples was very large. Deformation of the specimens may have occurred during the first test which caused the variation in the second resilient modulus value.
10. The unconfined creep-rebound test results indicated a strong relationship between the percentage of natural sand and rutting potential. The axial and permanent deformation values increased tremendously as the natural sand content increased. The creep modulus value decreased significantly as the percentage of natural sand increased. The creep-rebound test values were significantly affected at the 20 and 30 percent natural sand contents.
11. All laboratory test results indicated that asphalt concrete mixtures with all crushed aggregates had higher strength properties and would resist potential rutting better than mixtures containing natural sand materials. Asphalt concrete mixtures containing more than 20 percent natural sand appeared to have tremendous potential to deform under severe loads.

Recommendations

Based on the conclusions derived from the results of this laboratory study, the following recommendations were made:

1. To maximize the reduction in rutting potential for heavy duty pavements, all crushed aggregate should be used in the asphalt concrete mixture.
2. The maximum allowable limit for the natural sand content for heavy duty pavements should be less than 20 percent by weight. A conservative but practical maximum limit should be 15 percent natural sand.
3. Unconfined creep-rebound and indirect tensile tests should be used in conjunction with the Marshall procedure to analyze asphalt concrete mixtures in order to fully evaluate the engineering properties.
4. Aggregate gradations should be plotted on a gradation curve with the sieve sizes raised to the 0.45 power to evaluate the tenderness of the mixture.
5. Further laboratory studies should be conducted to evaluate the effects of other characteristics of natural sand materials in asphalt concrete mixtures. Aggregate type, angularity, particle shape, and gradation of the natural sand should be analyzed in more detail.
6. Field investigations should be conducted to verify field performance with laboratory data.

REFERENCES

1. Al-Hussaini, M. M., and Townsend, F C. (1973, May). "Tensile Testing of Soils -- A Literature Review, Miscellaneous Paper S-73-24, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

2. American Association of State Highway and Transportation Officials. (1988, November). "Report of AASHTO Joint Task Force on Rutting," Washington, DC

3. American Society of Testing and Materials. 1990. 1990 Annual Book of ASTM Standards, Vol 0403, Philadelphia, PA.

4. Anderton, G. L. (1990, January). "Evaluation of Asphalt Binder Modifiers," Miscellaneous Paper GL-90-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

5. Brown, E. R. (1974, December). "Evaluation of Asphalt Concrete Pavements Subjected to T-38 Aircraft Loadings," Thesis, Mississippi State University, Mississippi State, MS.

6. Brown, E. R. (1987, February). "Mix Design and Construction of Asphalt Concrete to Support High Tire Pressures," AASHTO/FHWA Symposium on High Pressure Truck Tires, Austin, TX.

7. Brown, E. R. and Foo, K. Y. (1989, October). "Evaluation of Variability in Resilient Modulus Test Results" (ASTM D 4123), ASTM Journal of Testing and Evaluation, Philadelphia, PA.

8. Button, J. W., and Perdomo, D. (1990, January). "Influence of Aggregate on Rutting in Asphalt Concrete Pavements," Transportation Research Board, Washington, DC.

9. Department of the Army. (1984, September). "Bituminous Intermediate and Wearing Courses for Airfields, Heliports and Heavy-Duty Pavements (Central-Plant Hot-Mix), Corps of Engineers Guide Specification, Military Construction CEGS-02556, Washington, DC.

10. Department of the Army and the Air Force. (1987, July). Technical Manual TM 5-822-8/AFM 88-6, Bituminous Pavements Standard Practice, Washington, DC.

11. Department of Commerce. (1962, May). "Aggregate Gradation for Highways," Washington, DC.

12. Department of Defense, MIL-STD-620A. (1966, January). Military Standard Test Methods of Bituminous Paving Materials, Washington, DC.

13. Development of Gyratory Testing Machine and Procedures for Testing Bituminous Paving Mixtures. (1962, February). Technical Report 3-595, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

14. Federal Highway Administration. (1988, March). "Asphalt Concrete Mix Design and Field Control," FHWA Technical Advisory T 5040.27, Washington, DC.

15. Foster, C. R. 1986. "The Effects of Voids in Mineral Aggregate on Pavement Performance," National Asphalt Pavement Association, Information Series 96/86, Riverdale, MD.

16. Grau, R. W. 1979. "Utilization of Marginal Construction Materials for LOC," Technical Report GL-79-21, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

17. Griffith, J. M., and Kallas, B. F. 1957. "Aggregate Voids Characteristics in Asphalt Paving Mixes," Highway Research Board Proceedings, Vol 36.

18. Griffith, J. M., and Kallas, B. F. 1958. "Influence of Fine Aggregates on Asphalt Concrete Paving Mixtures," Highway Research Board Proceedings, Vol 37.

19. Hanson, D. I. (1984, May). "Asphalt Mix Control in New Mexico," Proceedings FHWA Tri-Regional Pavement Rehabilitation Conference, Oklahoma City, OK.

20. Herrin, M. and Goetz, W. H. 1954. "Effect of Aggregate Shape on Stability of Bituminous Mixes," Highway Research Board Proceedings.

21. Investigation of the Design and control of Asphalt Paving Mixtures. (1948, May). Technical Memorandum No. 3-254, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

22. Kalcheff, I. V., and Tunnicliff, D. G. 1982. "Effects of Crushed Stone Aggregate Size and Shape on Properties of Asphalt Concrete," Proceedings AAPT, Vol 51.

23. Marker, V. (1977, June). "Tender Mixes: The Causes and Prevention," The Asphalt Institute, Information Series No. 168.

24. Marks, '. J., Monroe, R. W., and Adam, J. F. (1990, January). "The Effects of Crushed Particles in Asphalt Mixtures," Transportation Research Board, Washington, DC.

25. McRae, J. L. 1965. "Gyratory Testing Machine Technical Manual," Vicksburg, MS.

26. McRae, J. L. and Foster, C. R. (1959, April). "Theory and Application of a Gyratory Testing Machine for Hot-Mix Bituminous Pavement," Miscellaneous Paper 4-333, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

27. Page, G. C. (1984, May). "Rutting-Causes and Preventions," Proceedings FHWA Tri-Regional Pavement Rehabilitation Conference, Oklahoma City, OK.

28. Pavement Mix Design Study for Very Heavy Gear Loads Pilot Test Section. (1962, February). Technical Report 3-594, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

29. Shklarsky, E., and Livneh, M. 1964. "The Use of Gravels for Bituminous Mixtures," Proceedings AAPT, Vol 33.

30. Wedding, P. A., and Gaynor, R. D. 1961. "The Effects of Using Gravel as the Coarse and Fine Aggregate in Dense Graded Bituminous Mixtures," Proceedings AAPT, Vol 30.

31. ________________ (1982, April). "Rutting Investigation," Materials Laboratory, Wyoming State Highway Department, Cheyenne, WY.

32. ________________ 1984. "Asphalt Pavement Rutting-Western States," Western Association of State Highway and Transportation Officials, WASHTO Executive Committee.

APPENDIX A: UNCONFINED CREEP-REBOUND CURVES

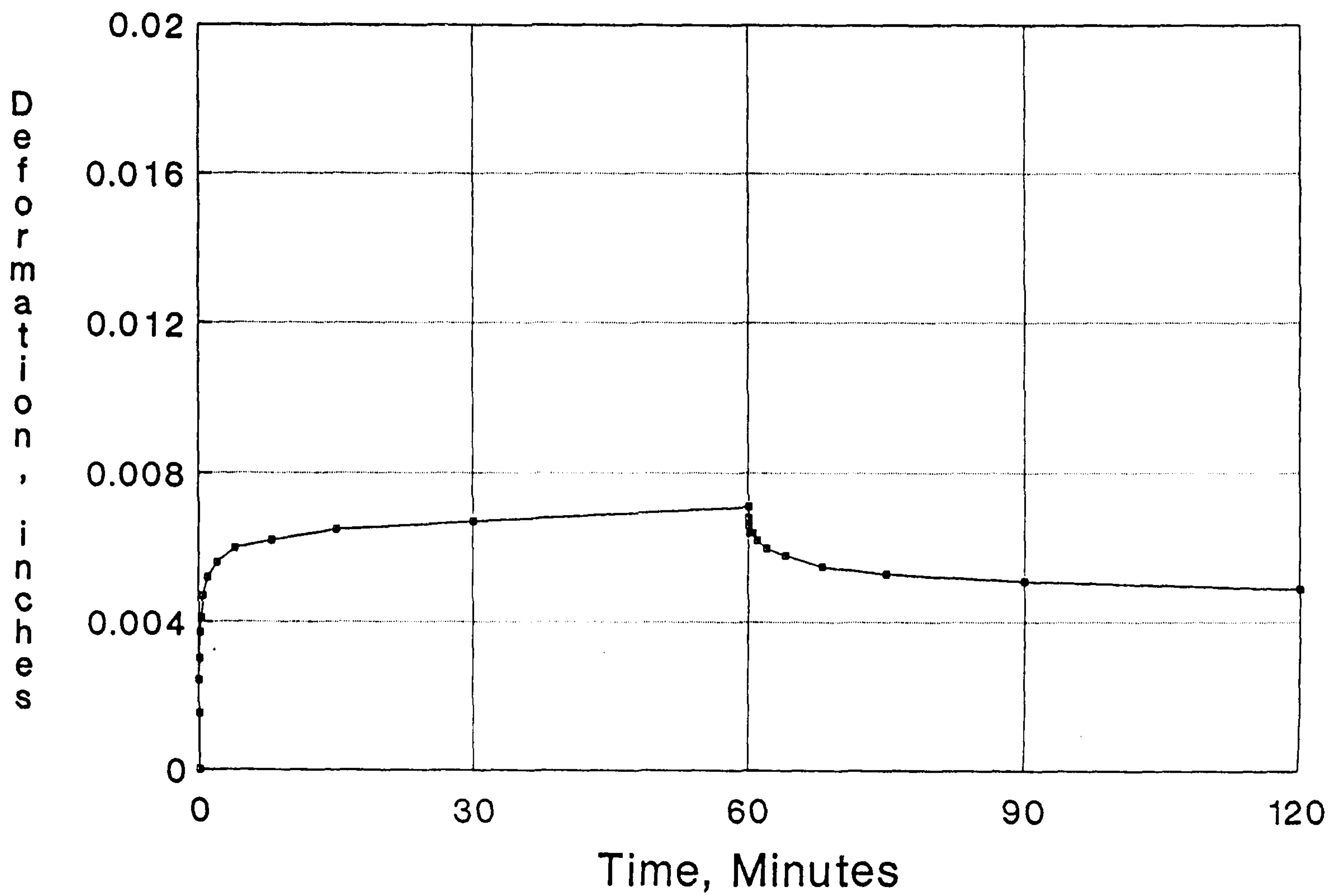


Figure 50. Creep-Rebound Curve for S-01 Sample at 77°F

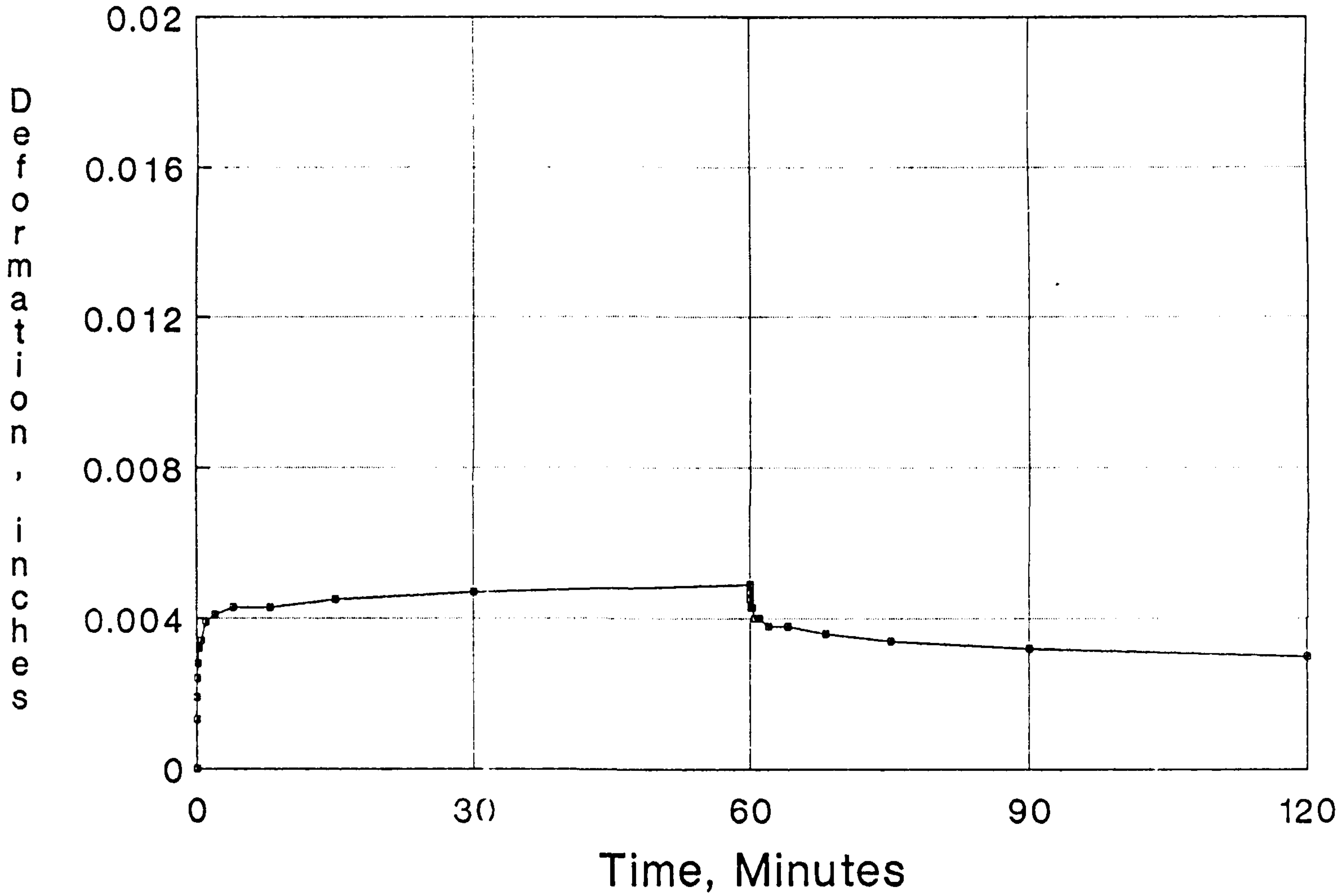


Figure 51. Creep-Rebound Curve for S-02 Sample at 77°F

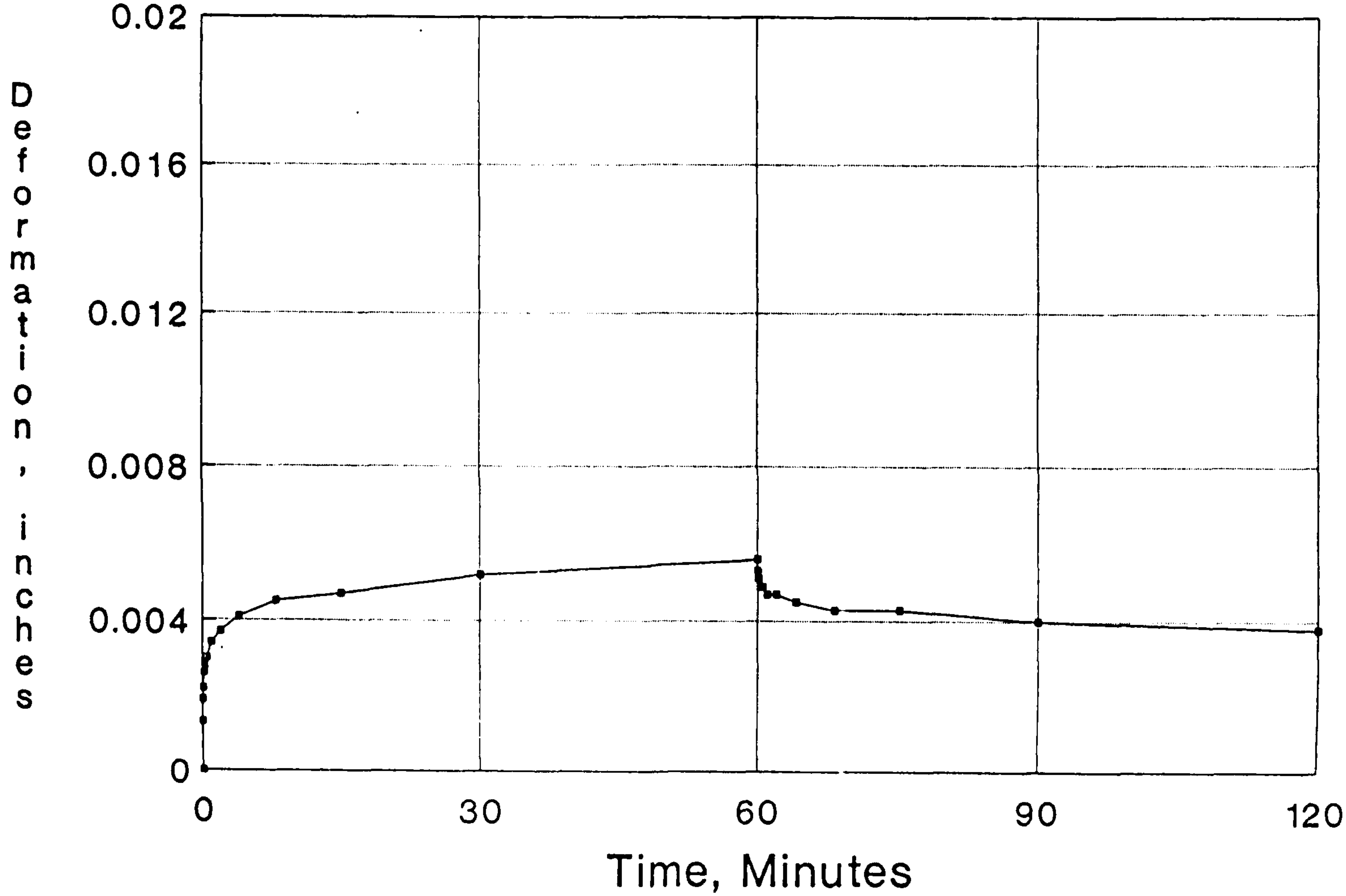


Figure 52. Creep-Rebound Curve for S-03 Sample at 77°F

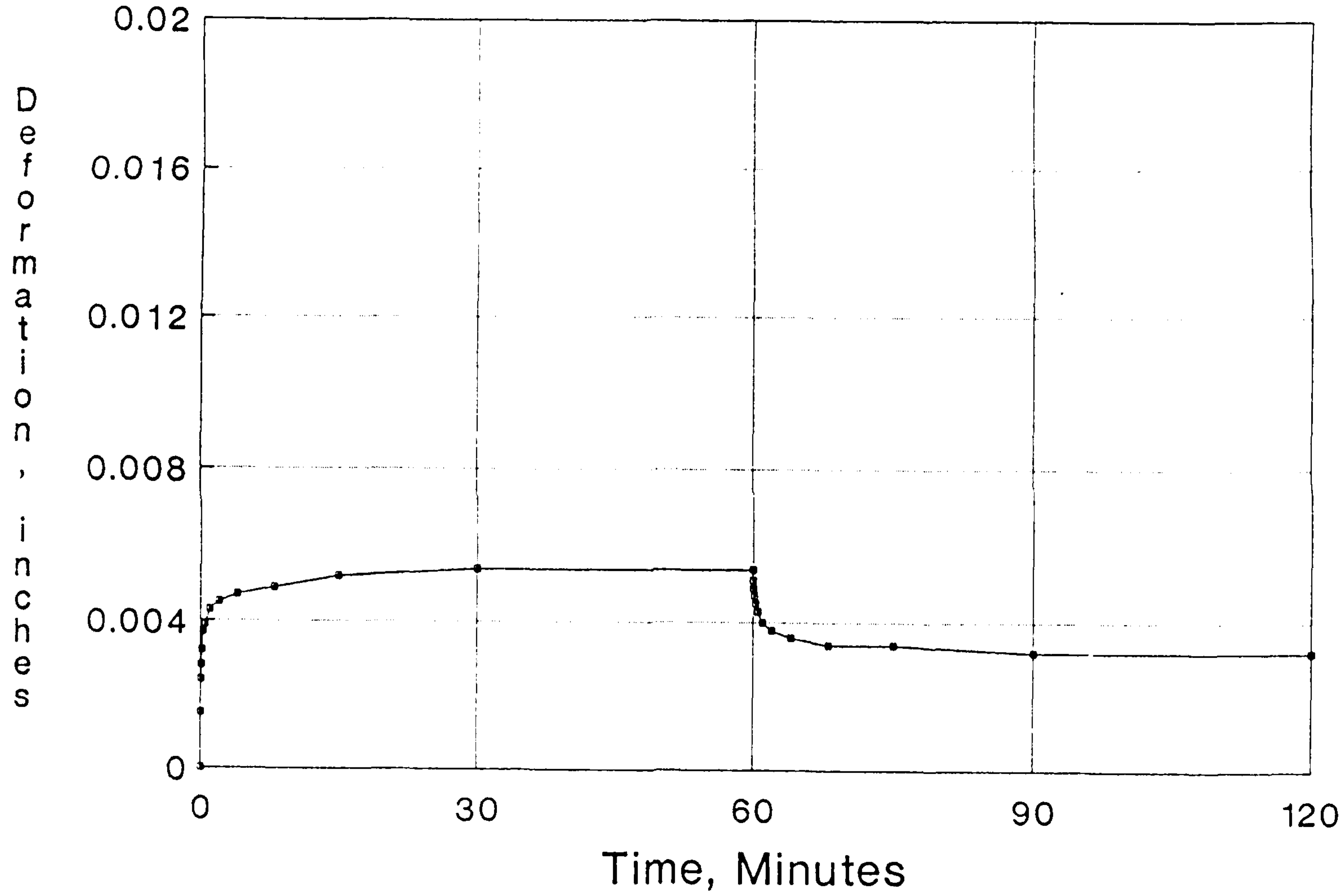


Figure 53. Creep-Rebound Cruve for S-04 Sample at 104°F

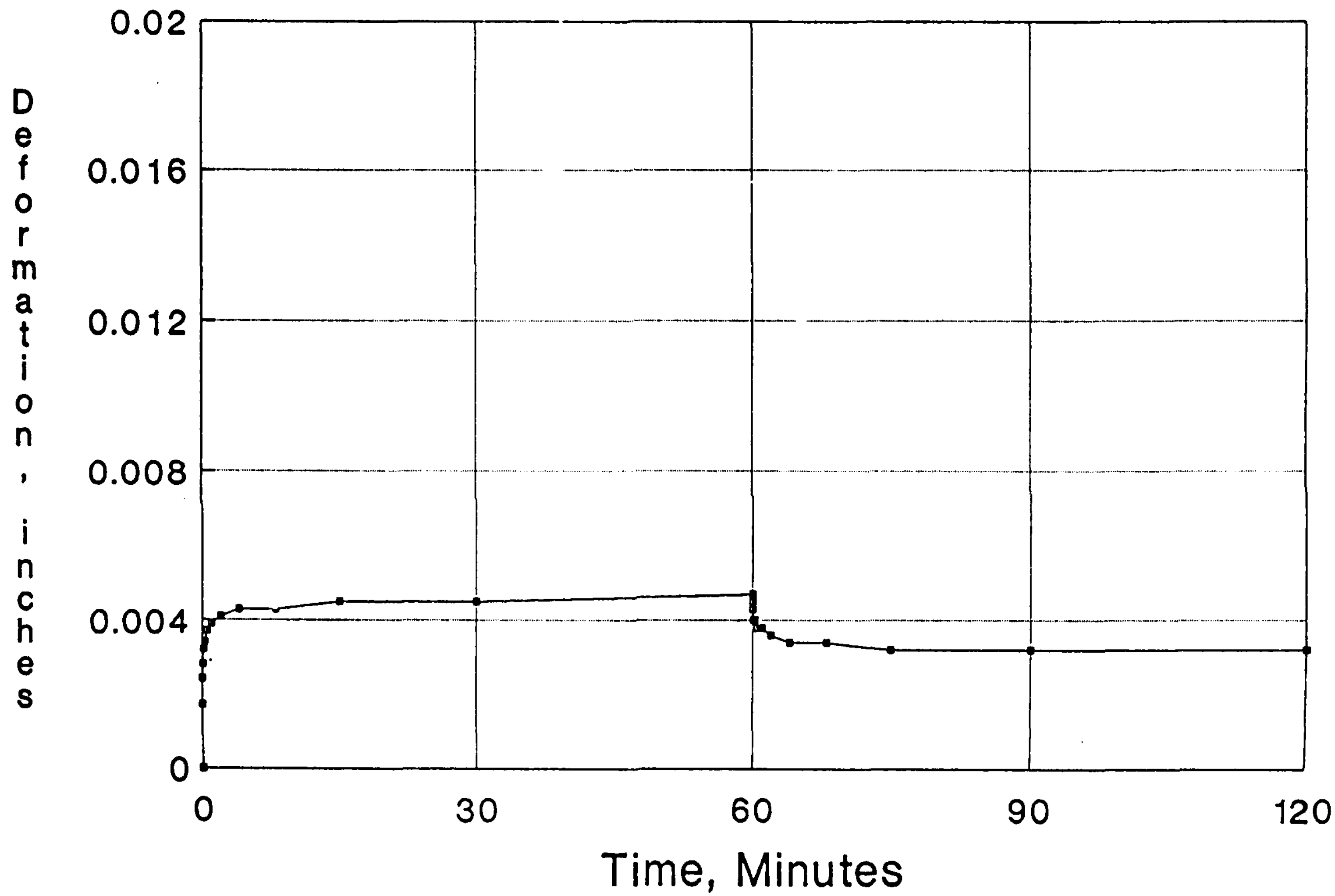


Figure 54. Creep-Rebound Curve for S-05 Sample at 104°F

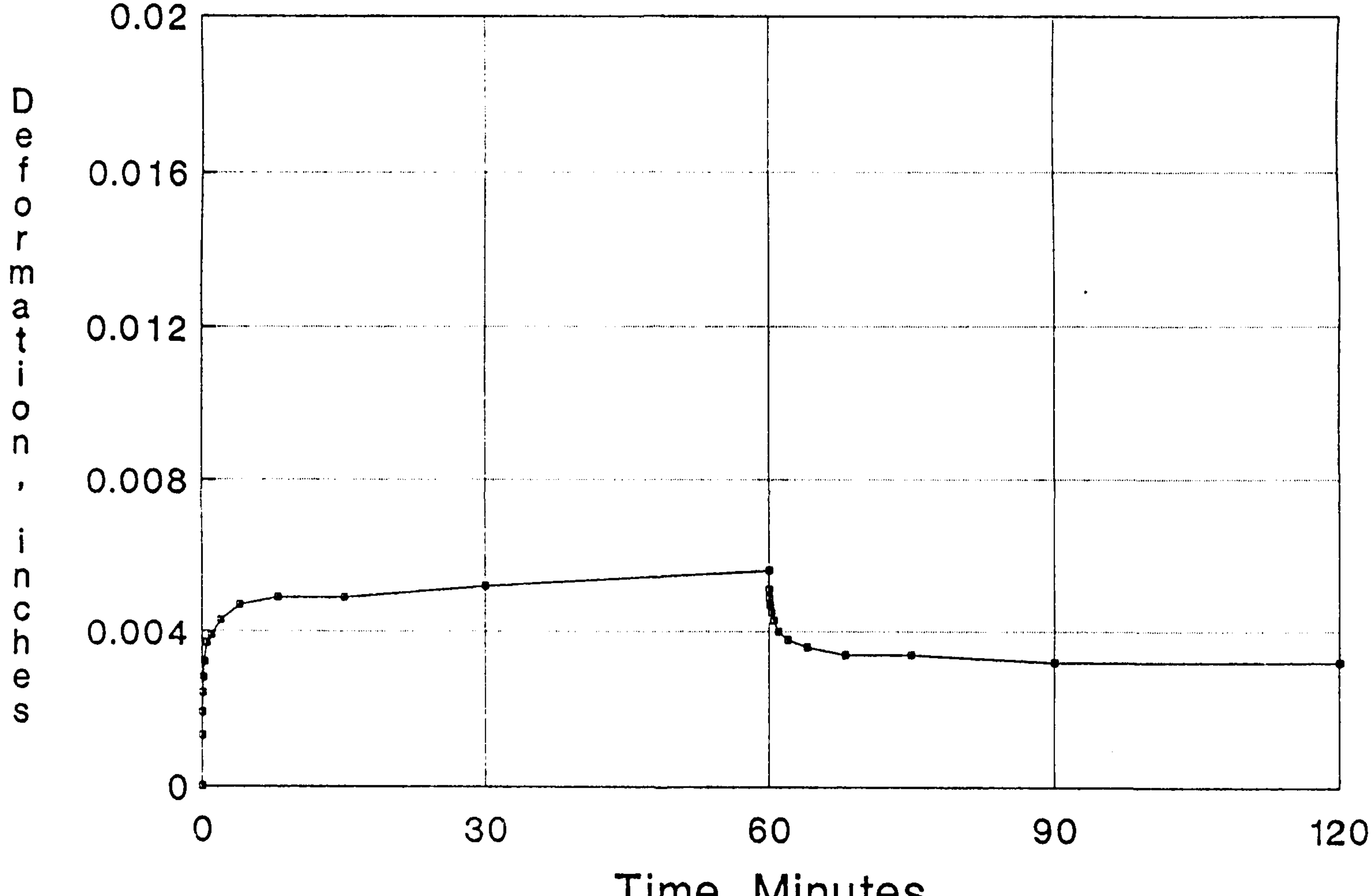


Figure 55. Creep-Rebound Curve for S-06 Sample at 104°F

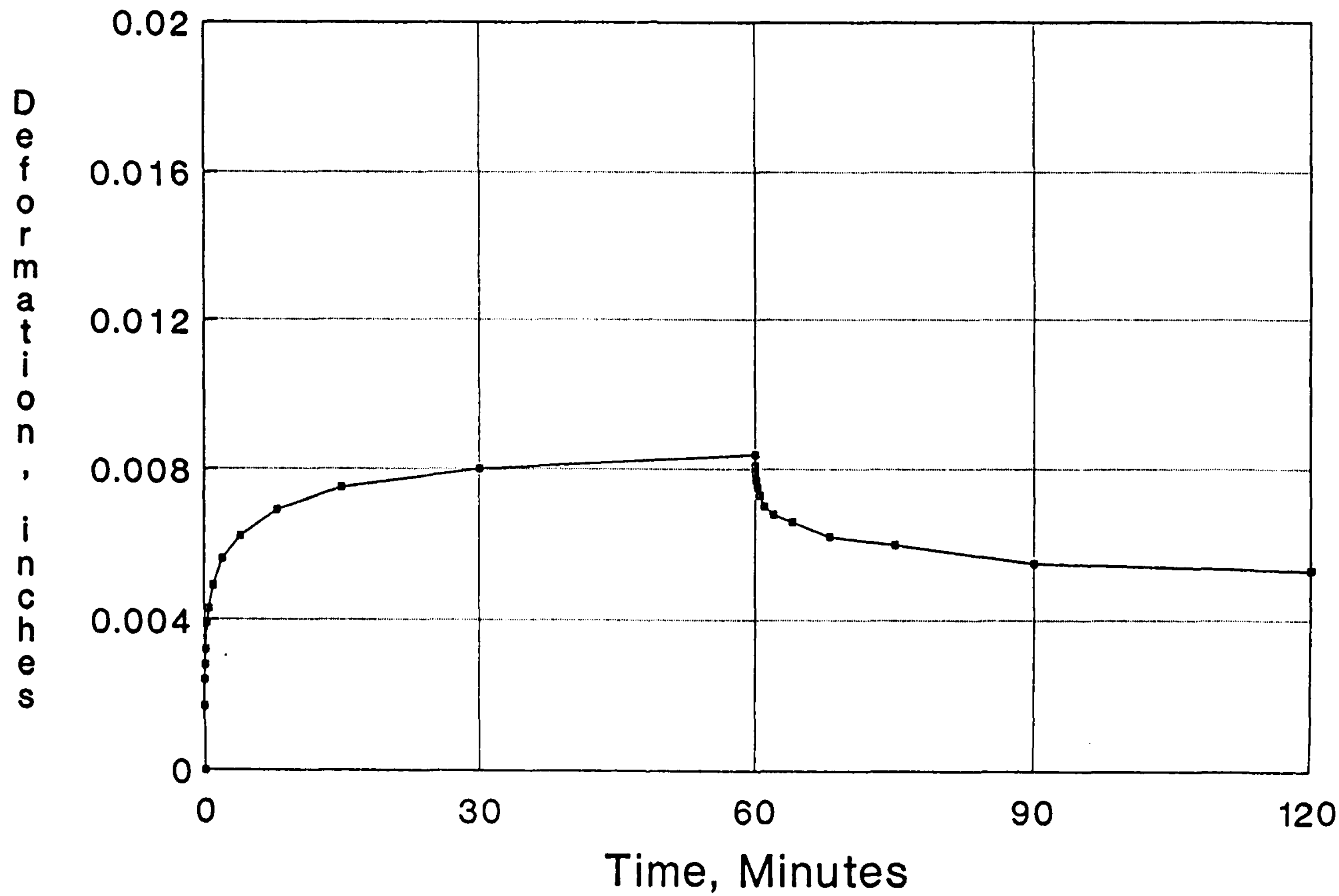


Figure 56. Creep-Rebound Curve for S-1M1 Sample at 77°F

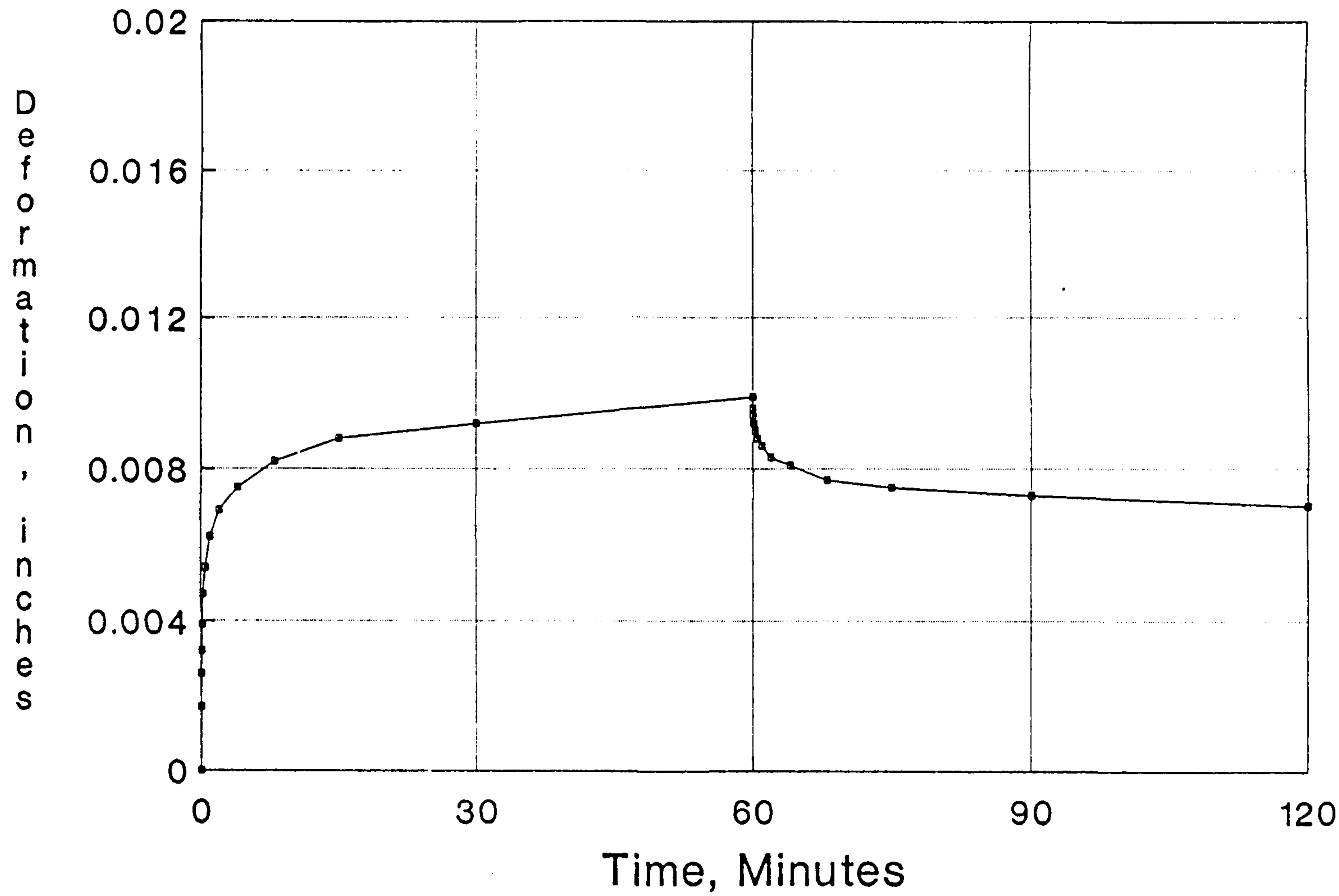


Figure 57. Creep-Rebound Curve for S-1M2 Sample at 77°F

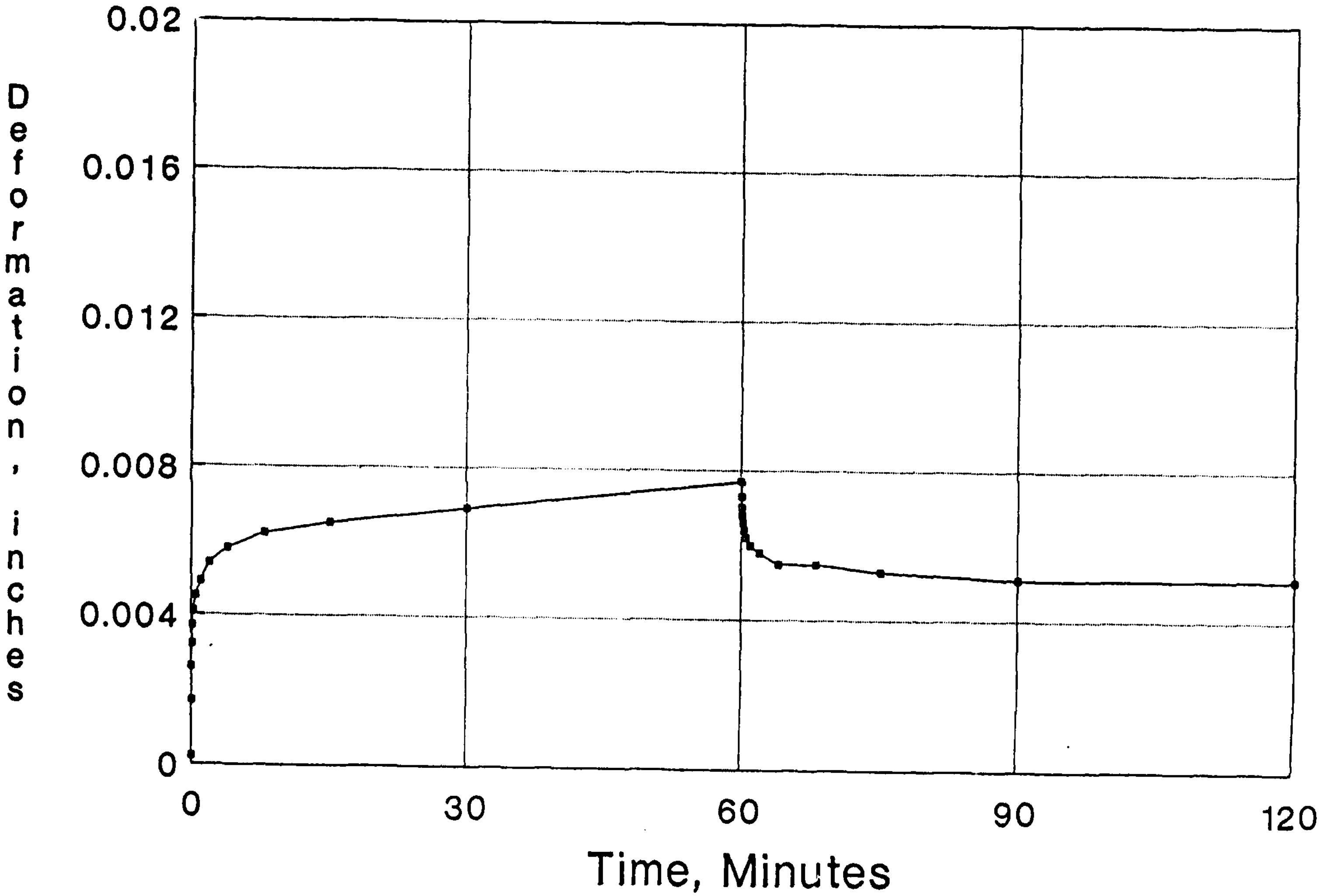


Figure 58. Creep-Rebound Curve for S-1M3 Sample at 104°F

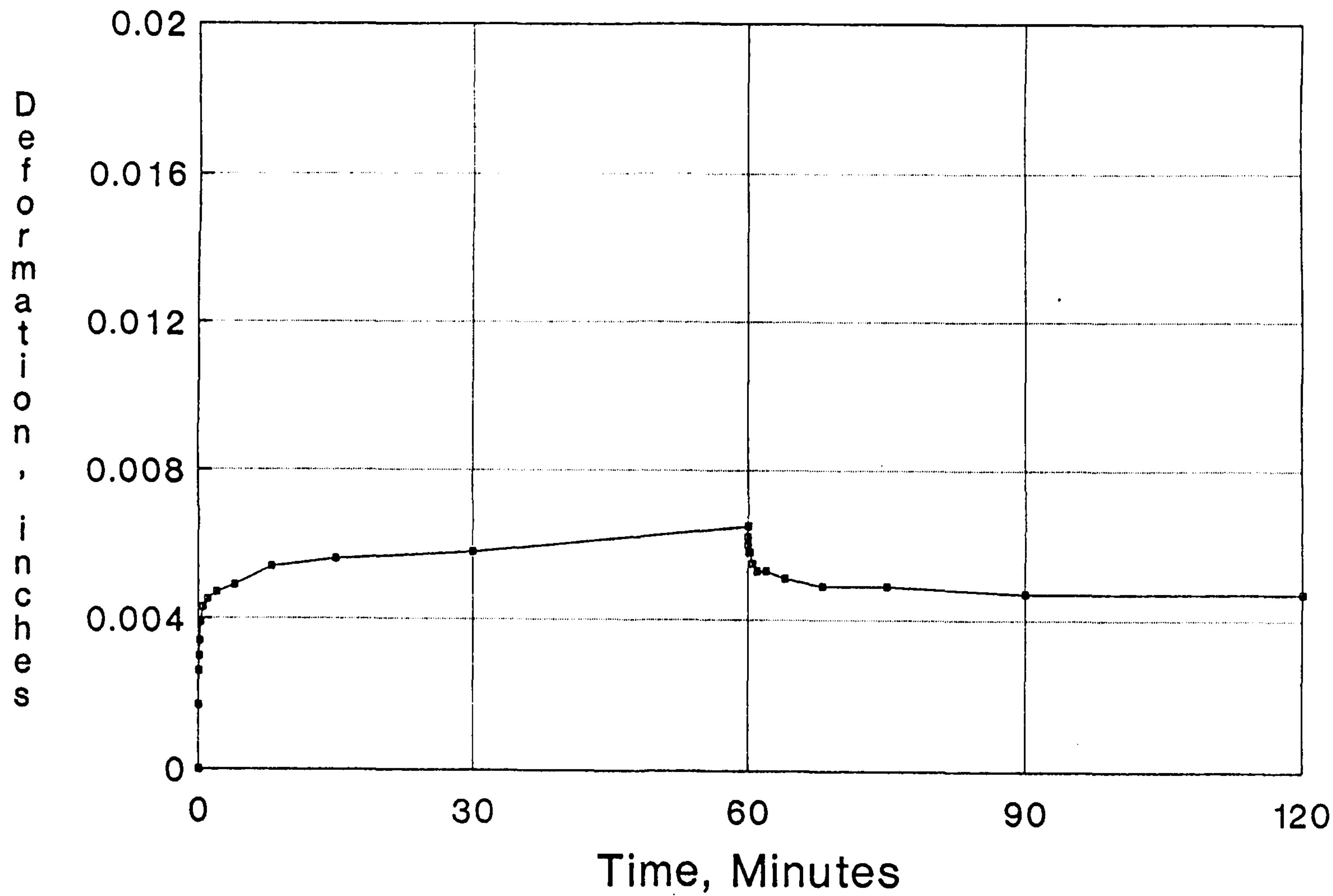


Figure 59. Creep-Rebound Curve for S-1M4 Sample at 104°F

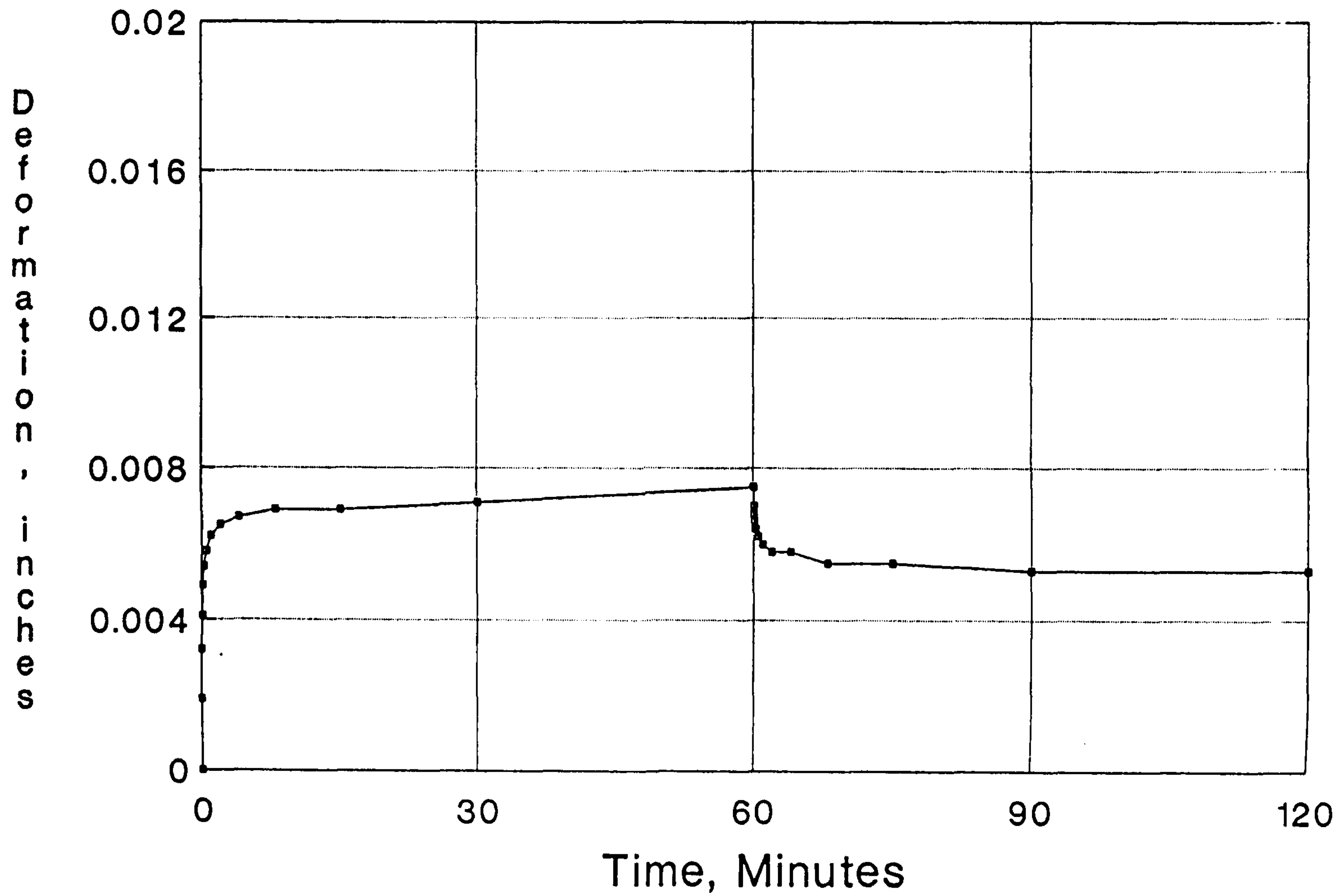


Figure 60. Creep-Rebound Curve for S-1M5 Sample at 104°F

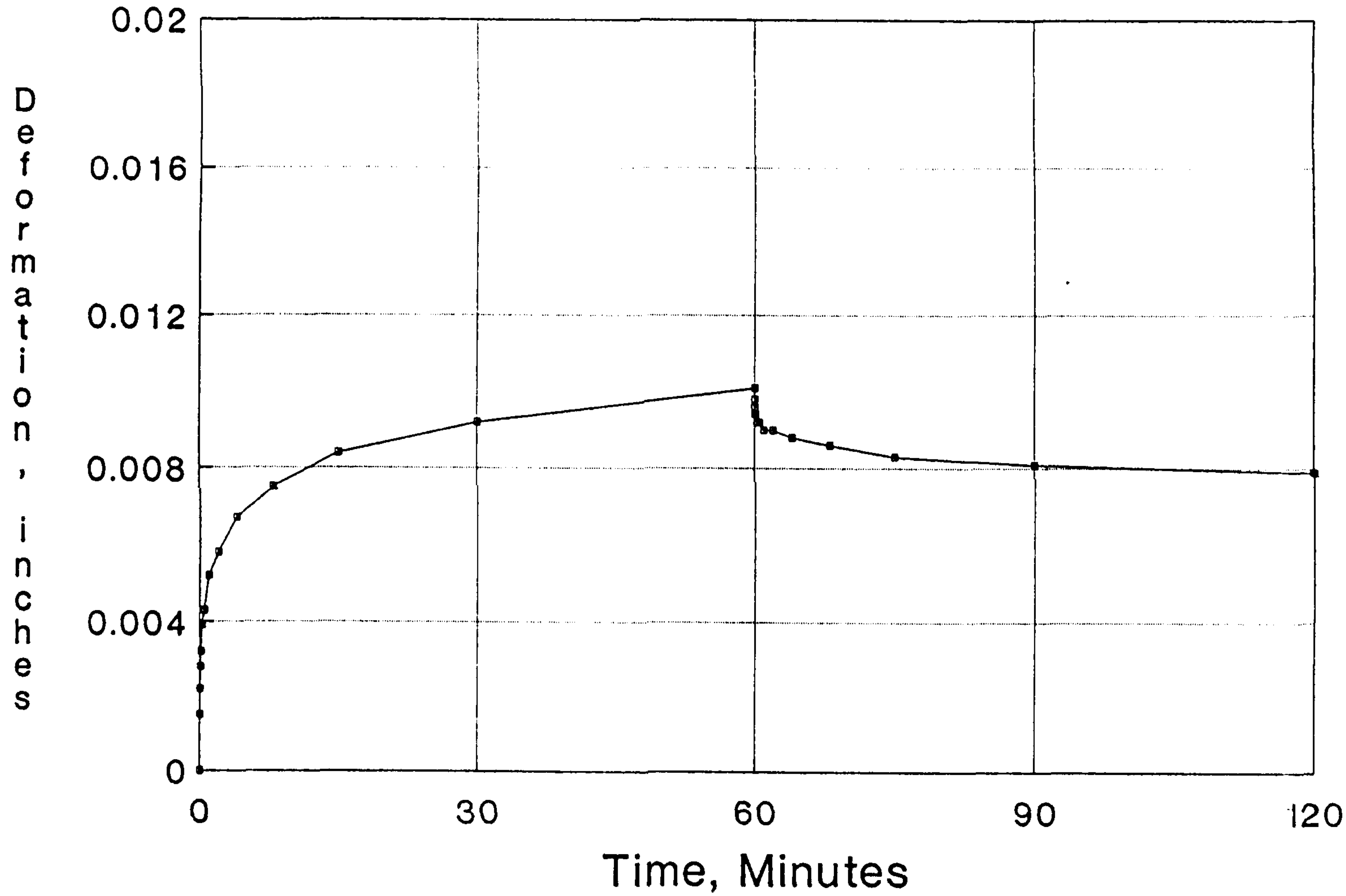


Figure 61. Creep-Rebound Curve for S-2M1 Sample at 77°F

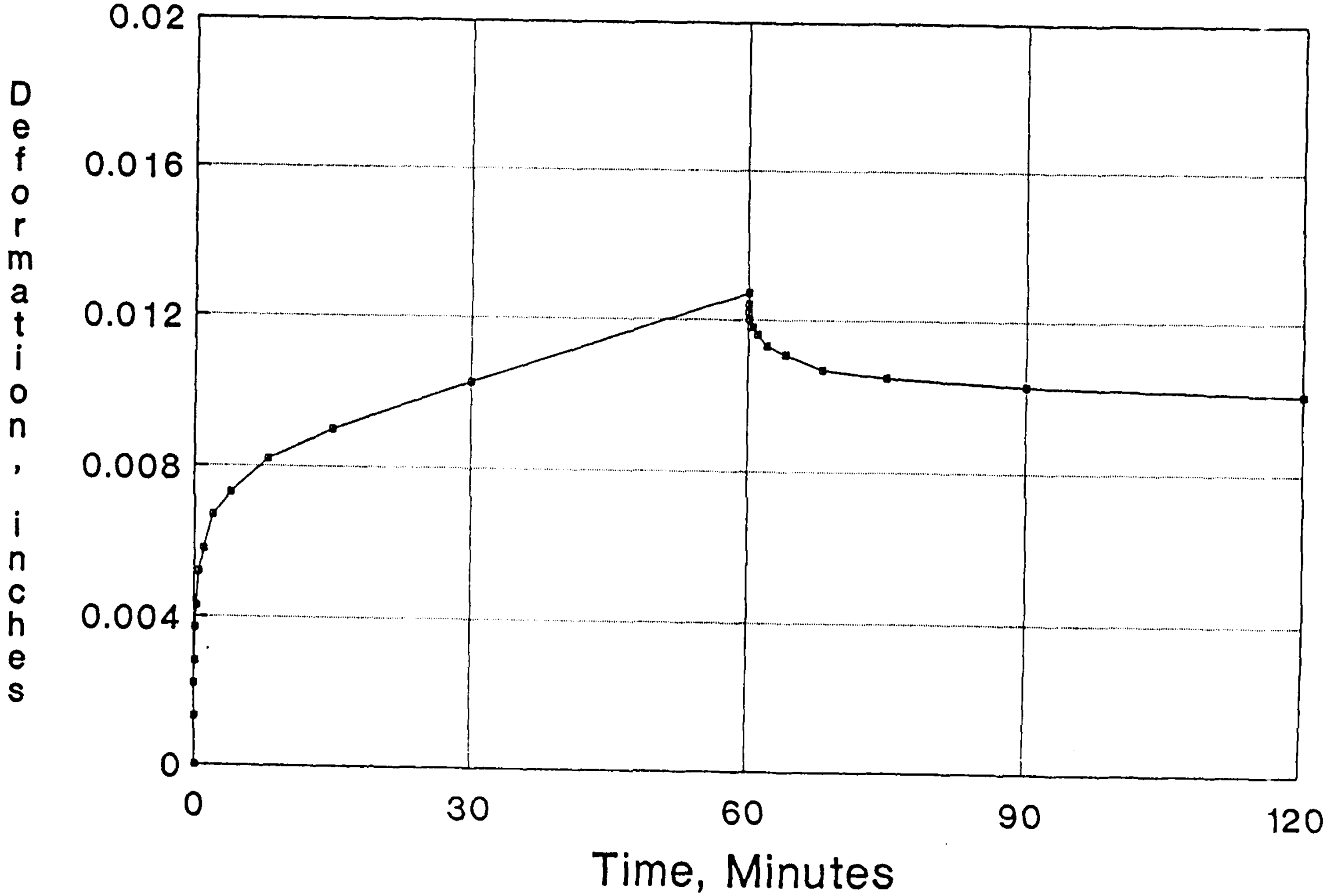


Figure 62. Creep-Rebound Curve for S-2M2 Sample at 77°F

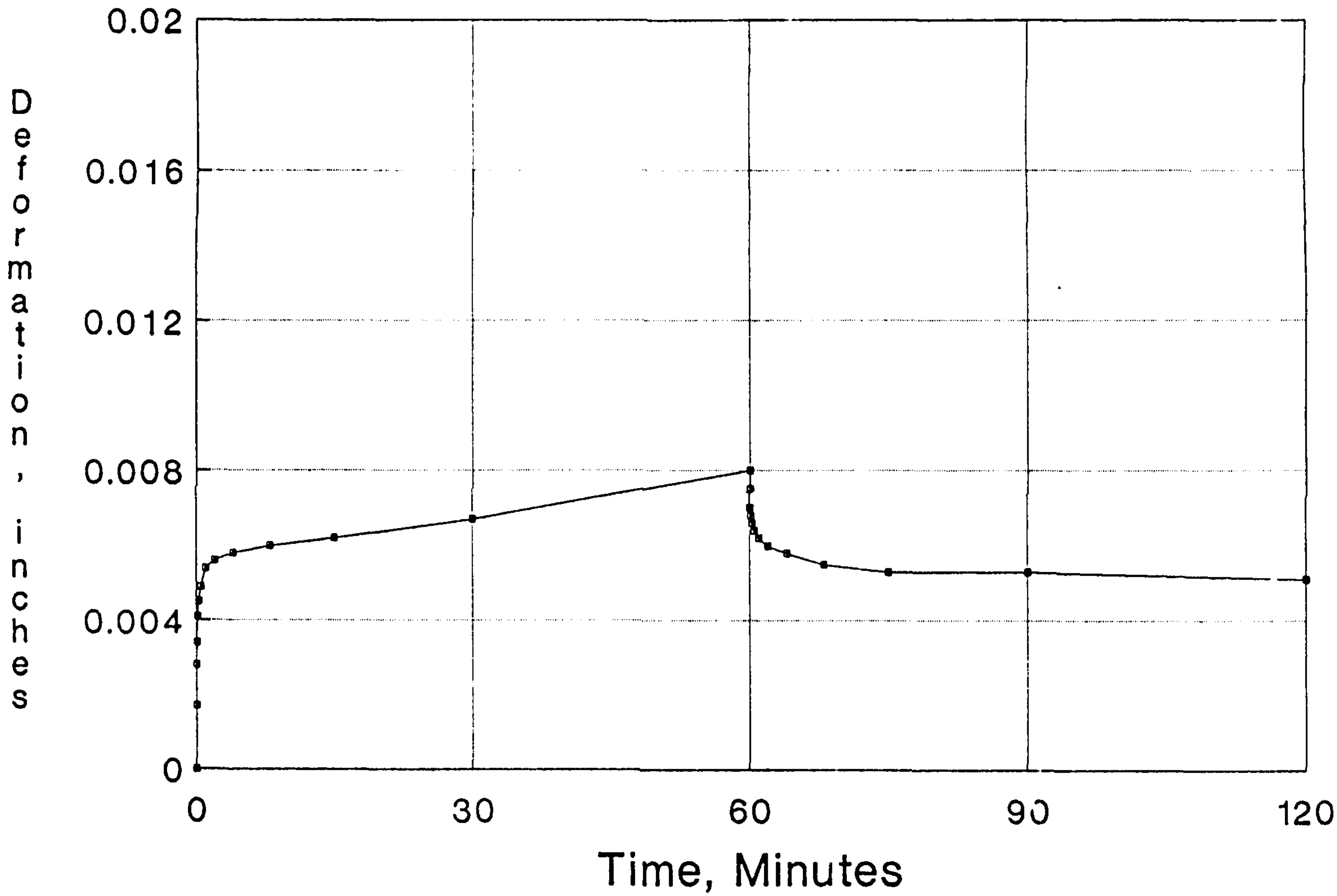


Figure 63. Creep-Rebound Curve for S-2M3 Sample at 104°F

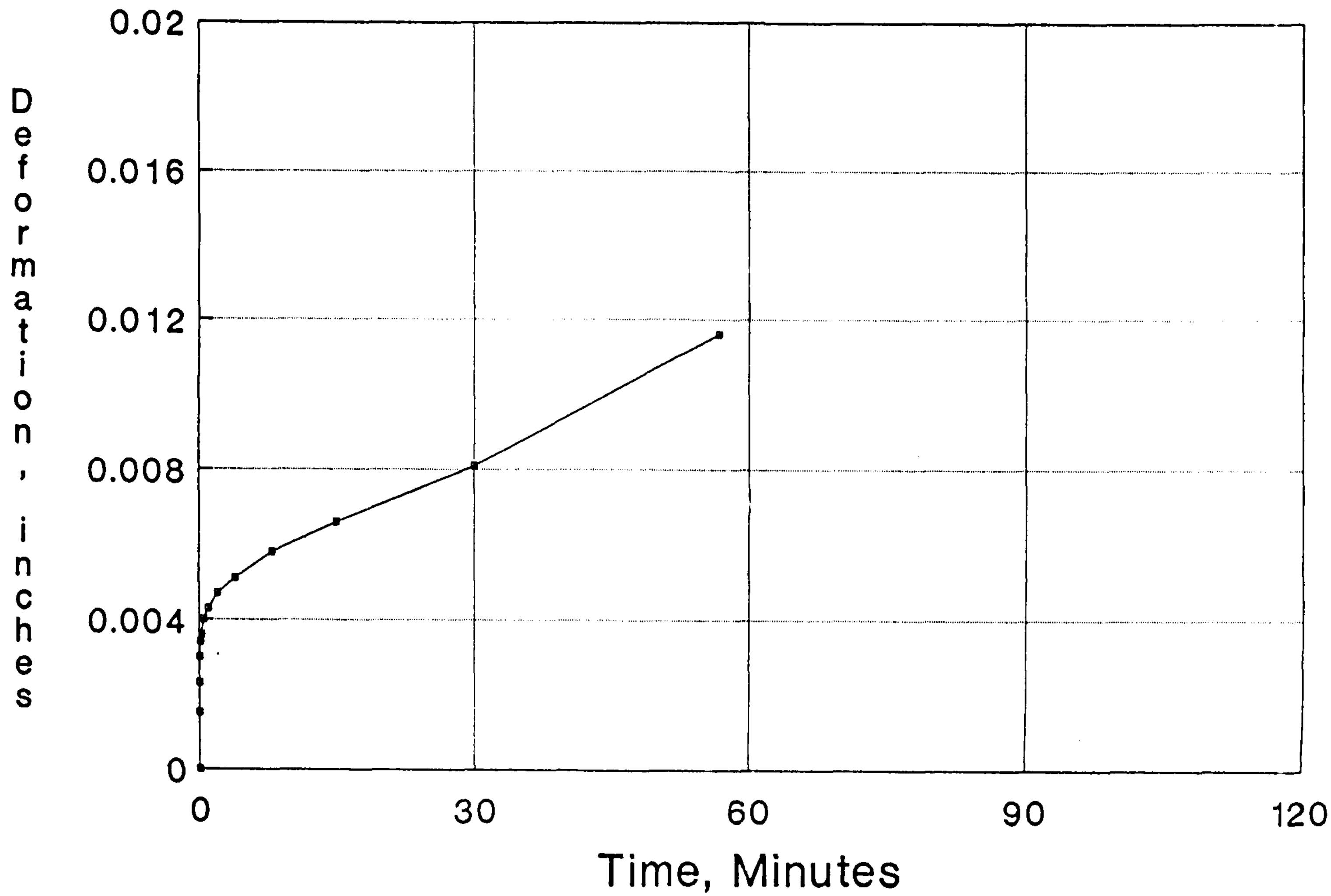


Figure 64. Creep-Rebound Curve for S-2M4 Sample at 104°F

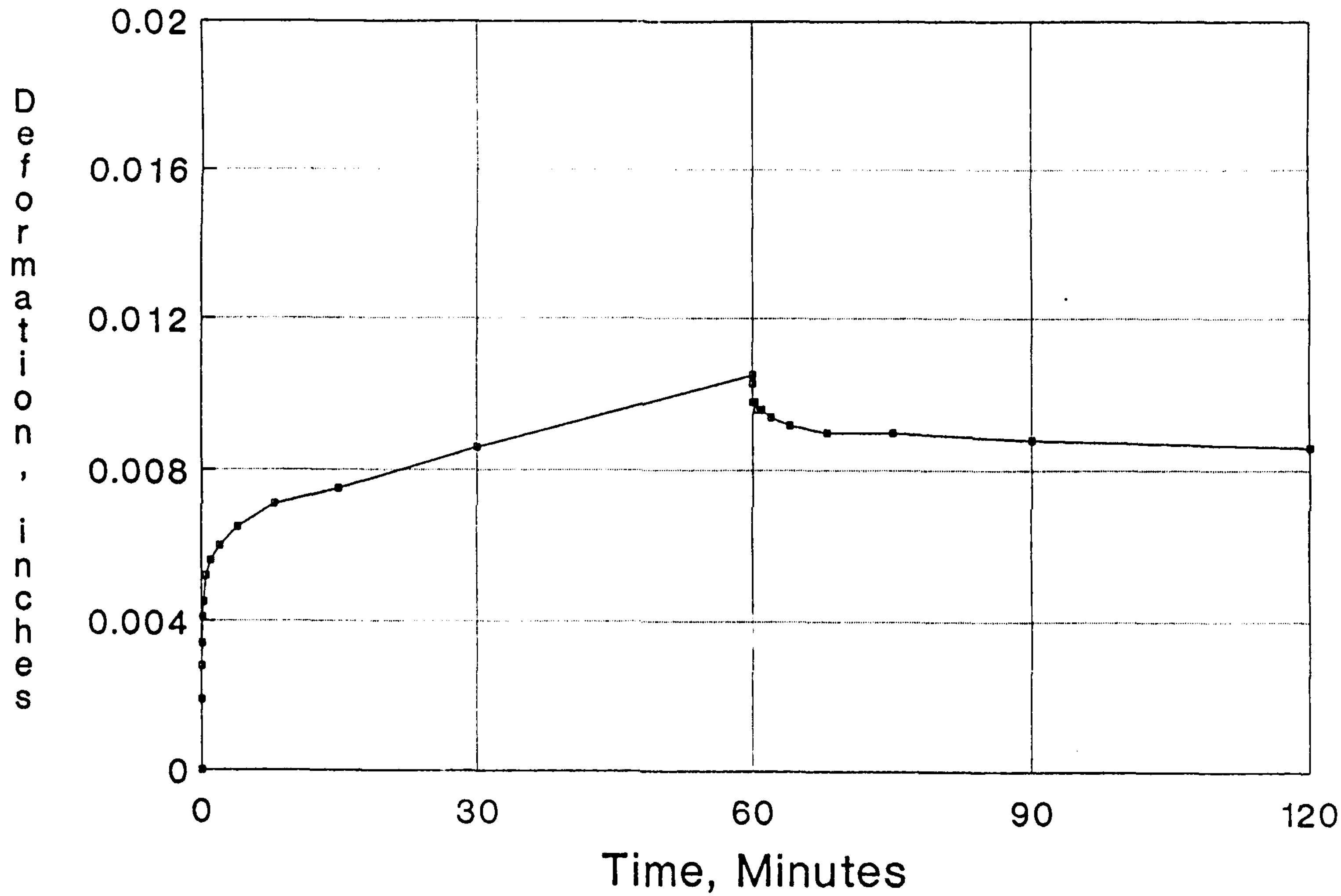


Figure 65. Creep-Rebound Curve for S-2M5 Sample at 104°F

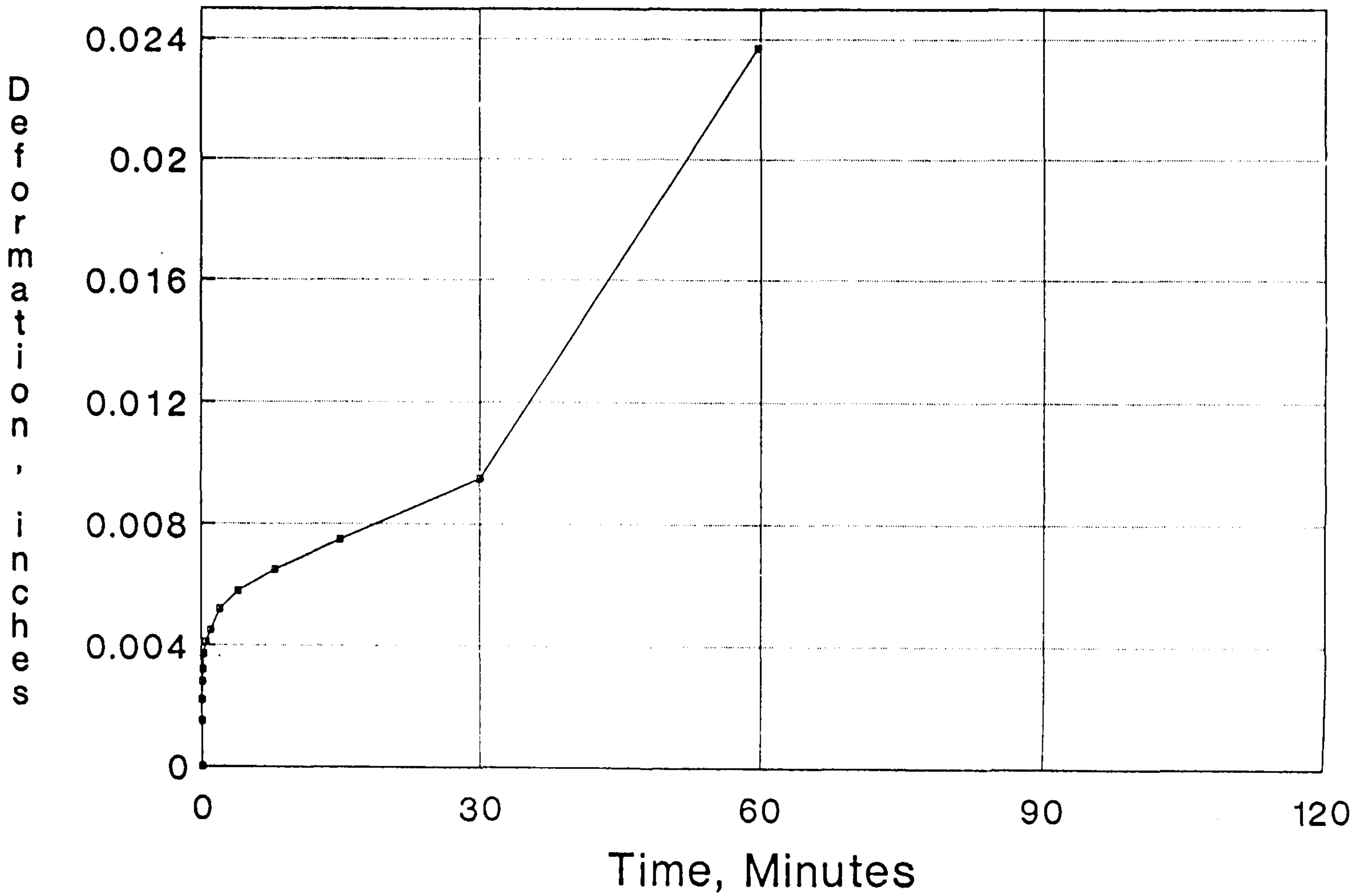


Figure 66. Creep-Rebound Curve for S-2M6 Sample at 104°F

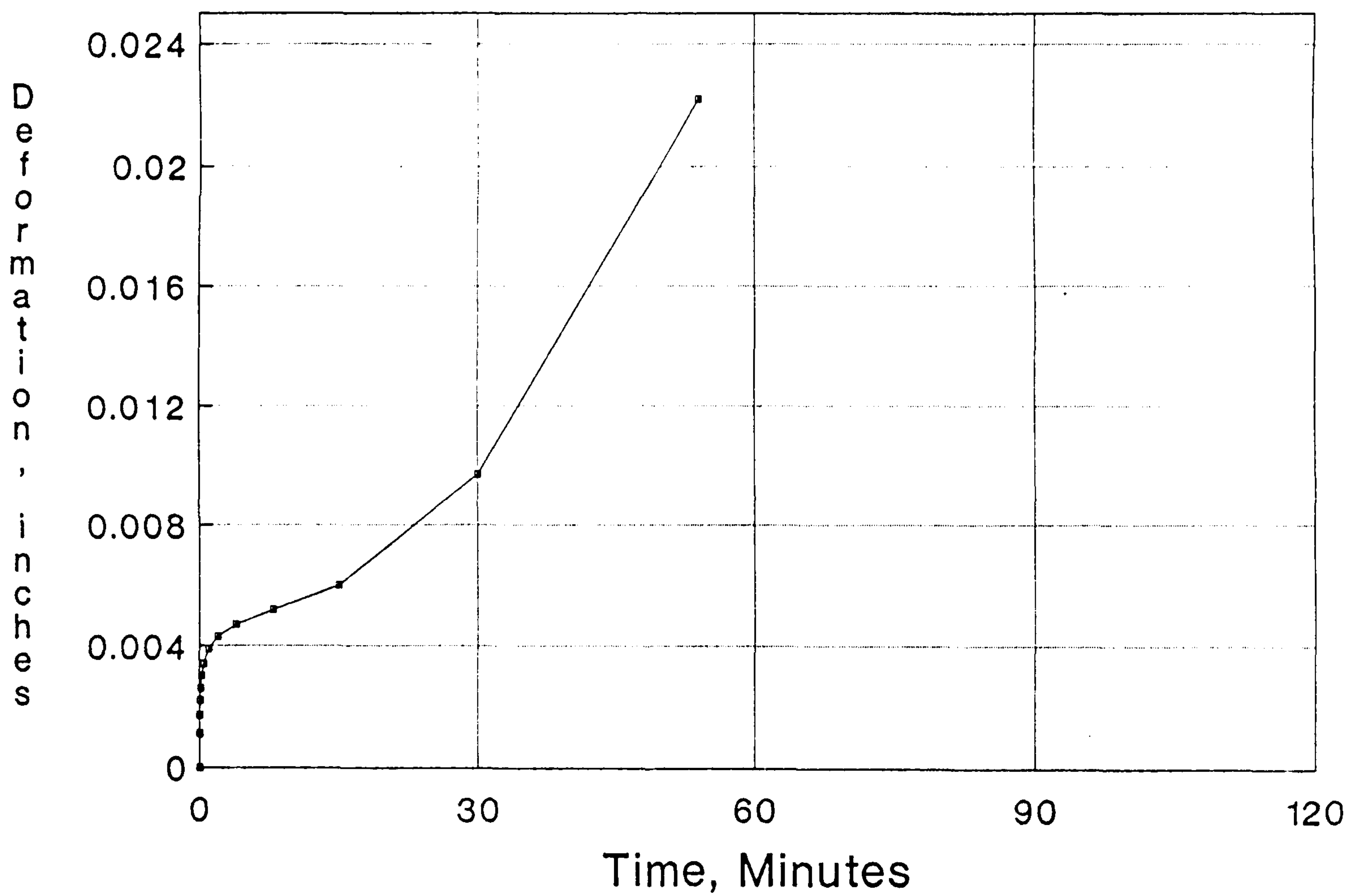


Figure 67. Creep-Rebound Curve for S-3M1 for 77°F

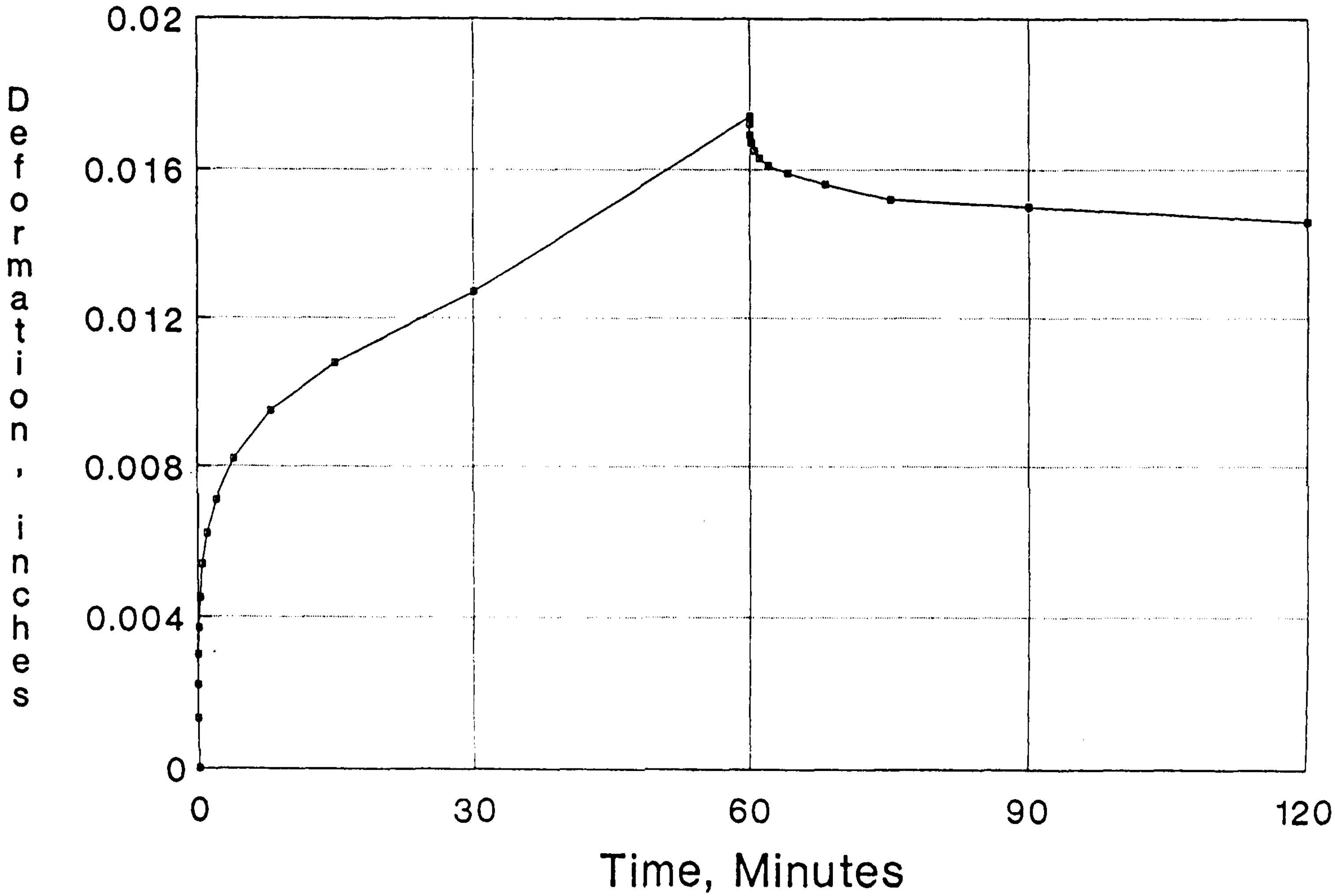


Figure 68. Creep-Rebound Curve for S-3M2 Sample at 77°F

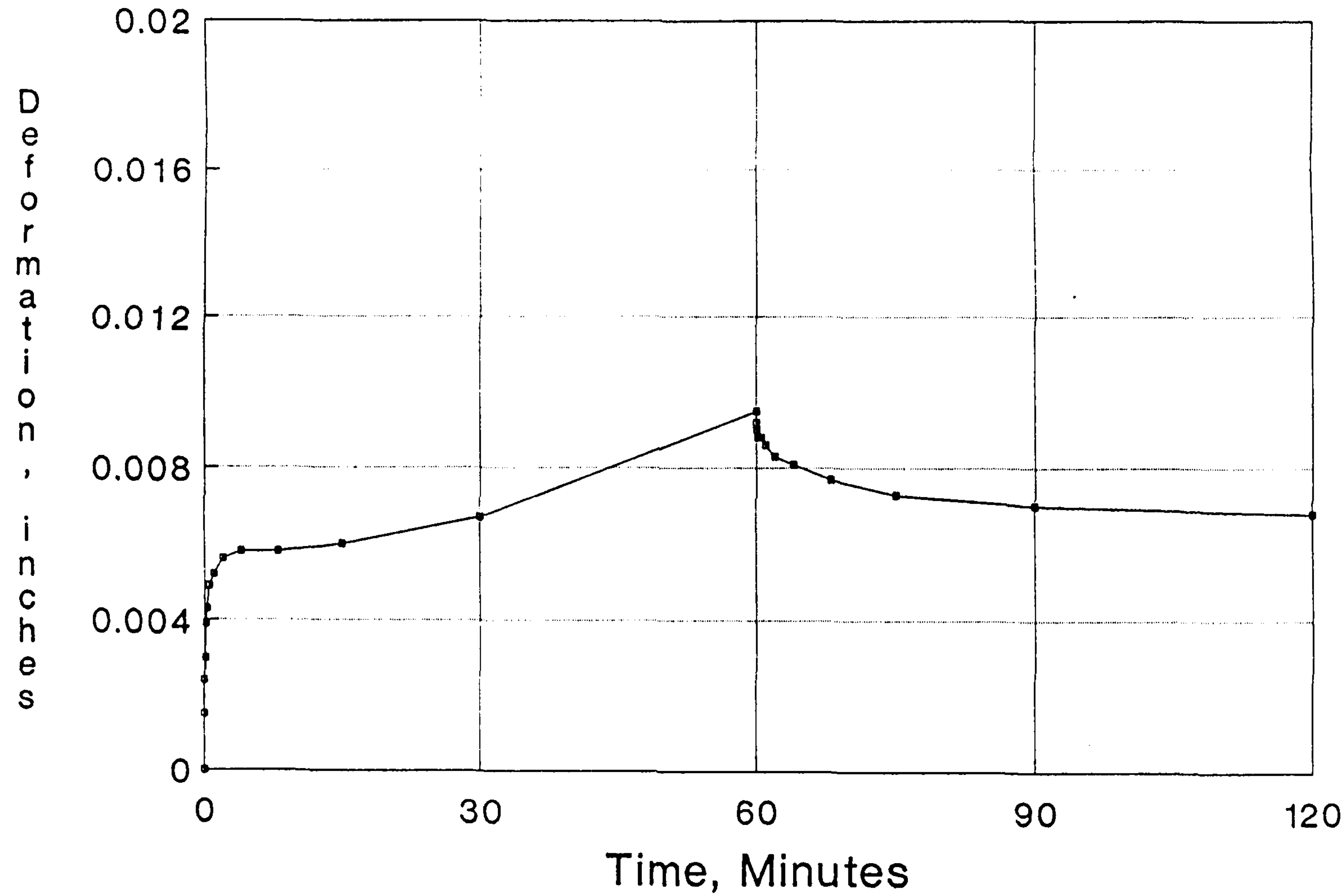


Figure 69. Creep-Rebound Curve for S-3M3 Sample at 77°F

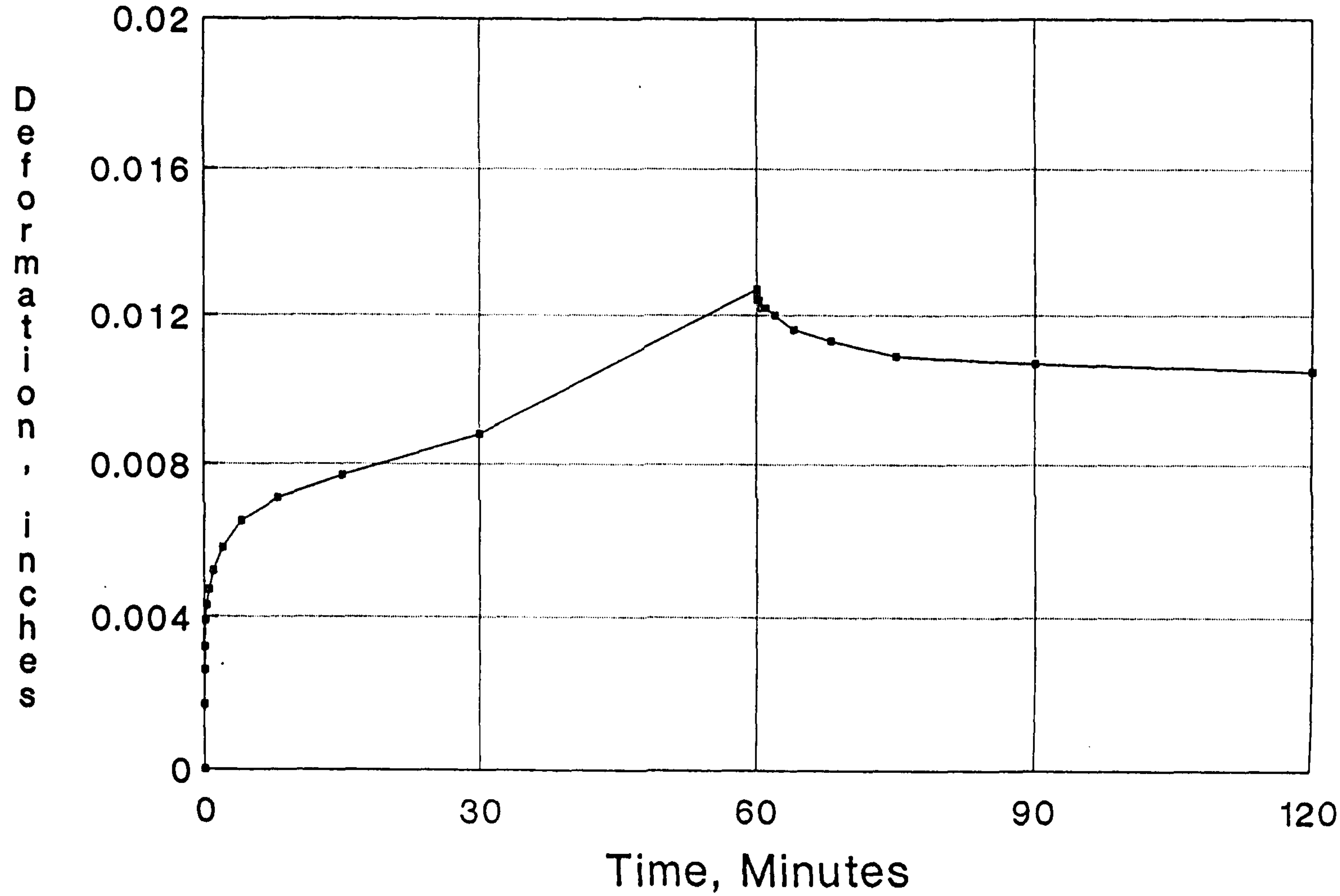


Figure 70. Creep-Rebound Curve for S-3M4 Sample at 77°F

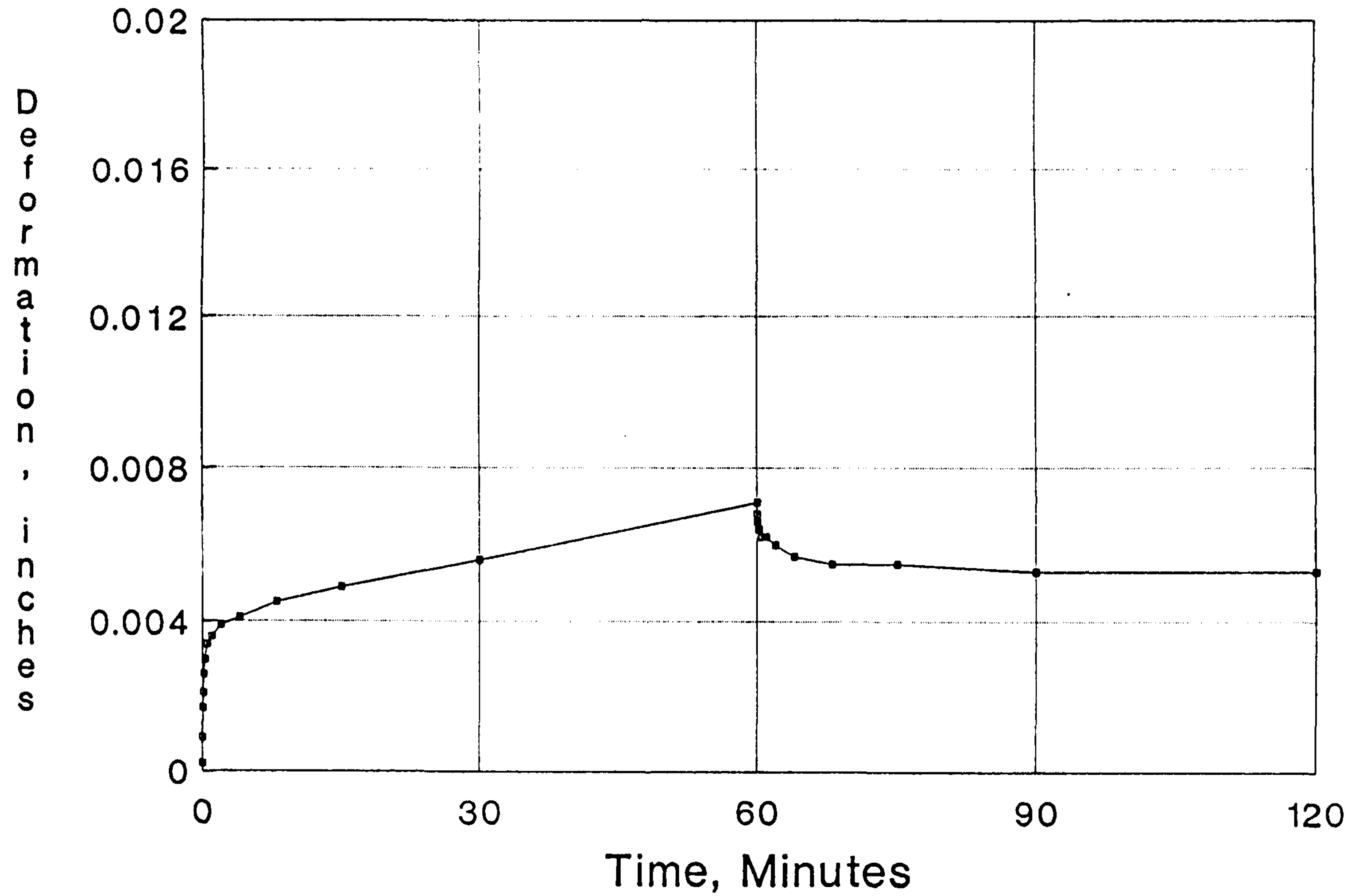


Figure 71. Creep-Rebound Curve for S-3M5 Sample at 104°F

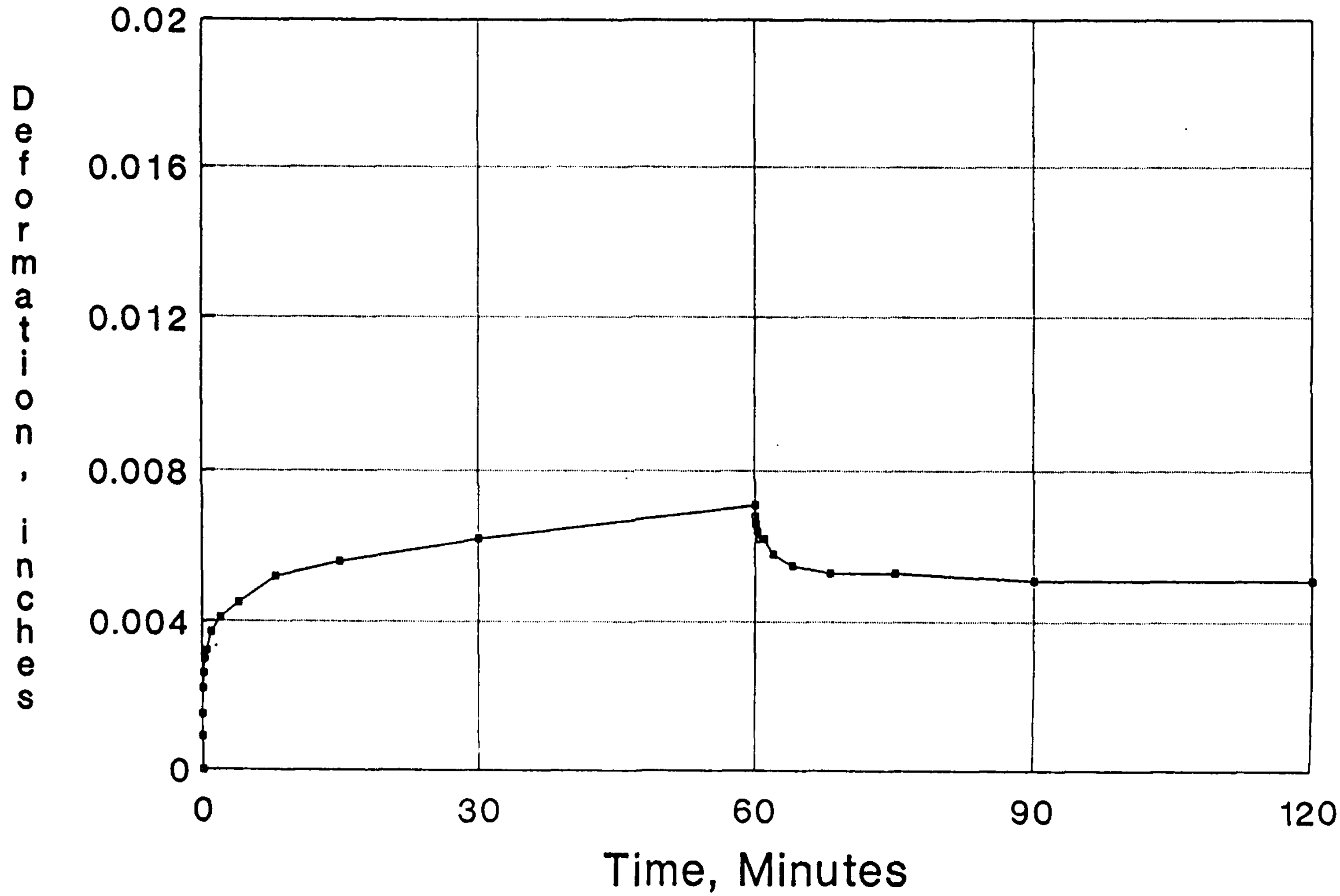


Figure 72. Creep-Rebound Curve for S-3M6 Sample at 104°F

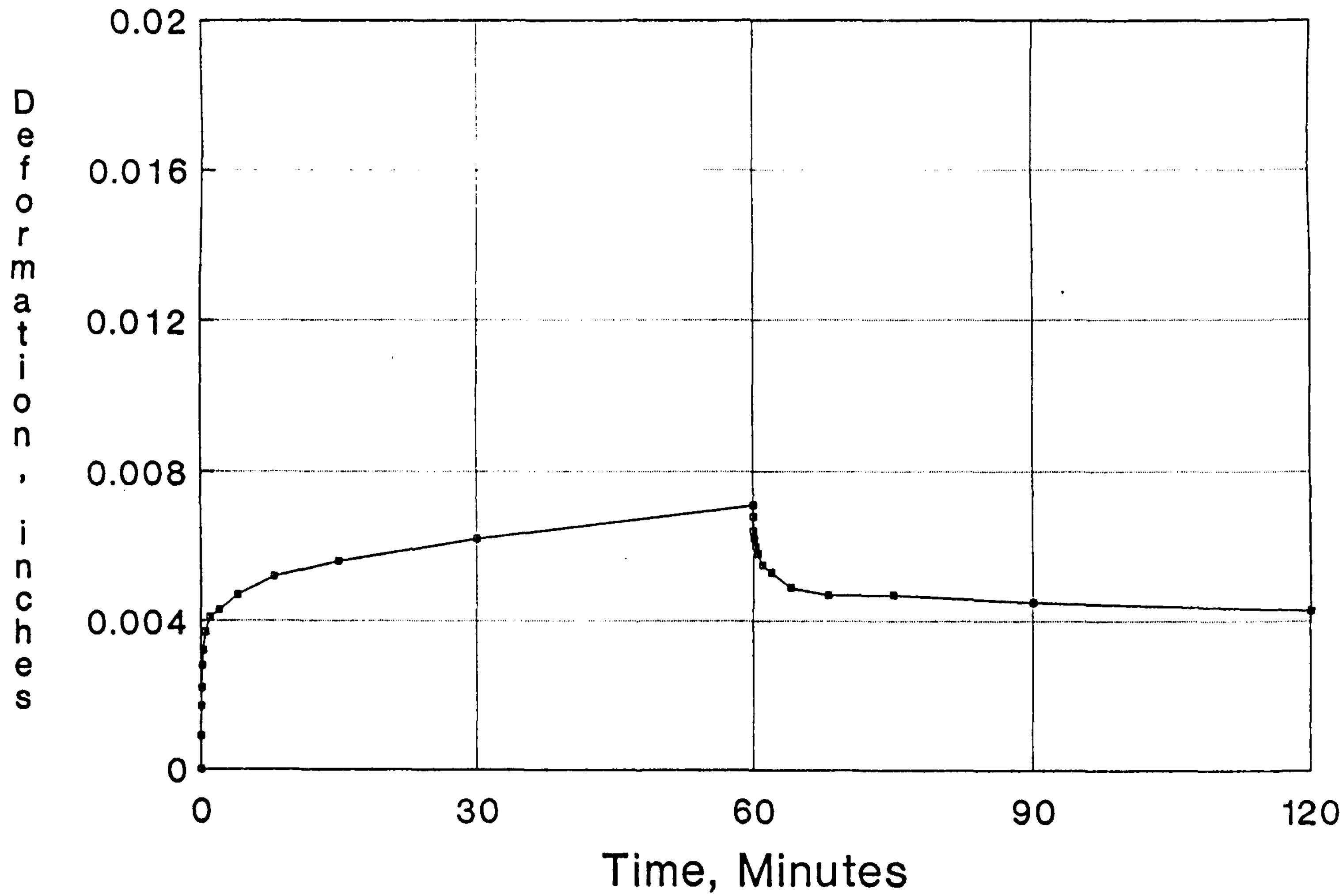


Figure 73. Creep-Rebound Curve for S-3M7 Sample at 104°F

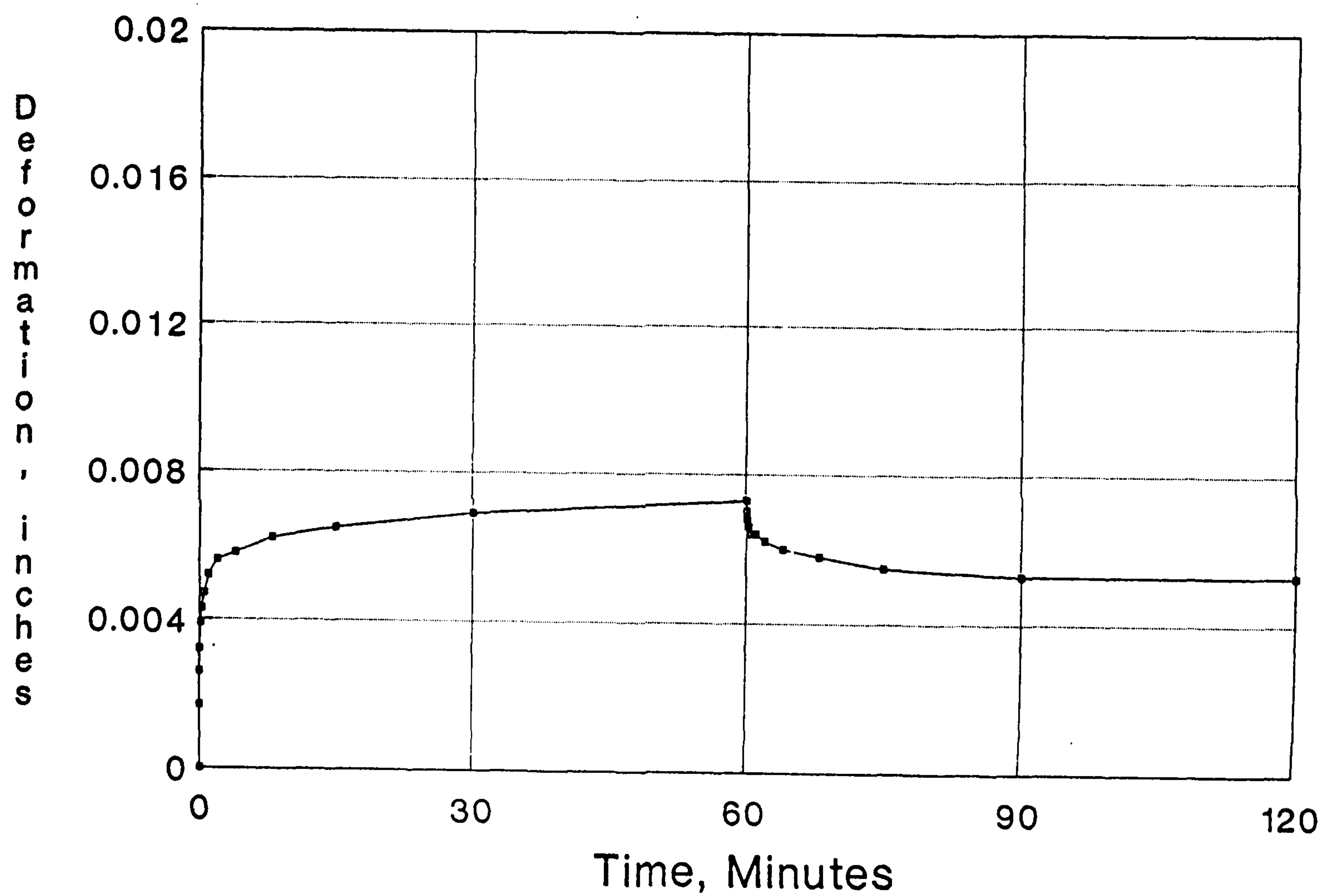


Figure 74. Creep-Rebound Curve for S-1C1 Sample at 77°F

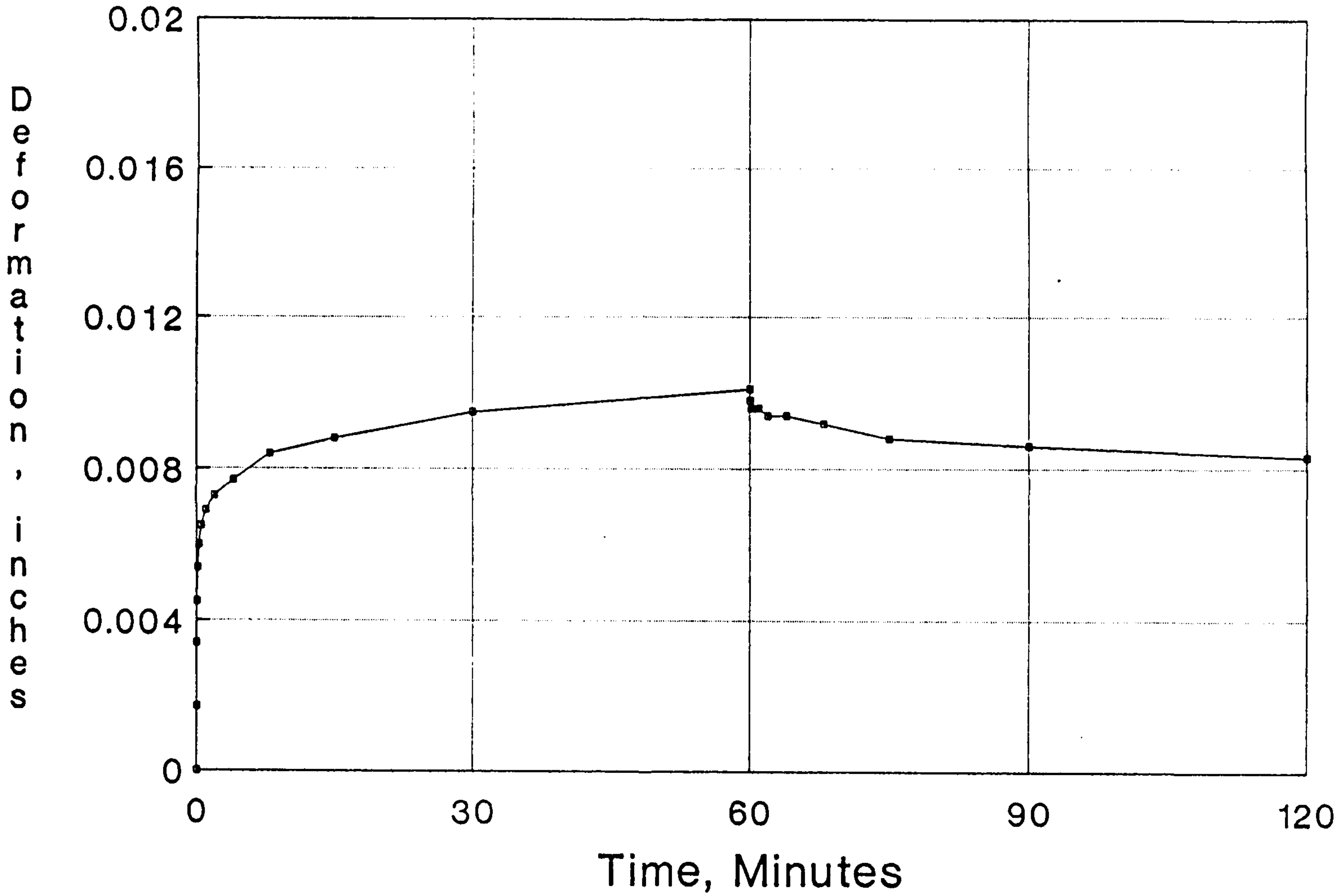


Figure 75. Creep-Rebcund Curve for S-1C2 Sample at 77°F

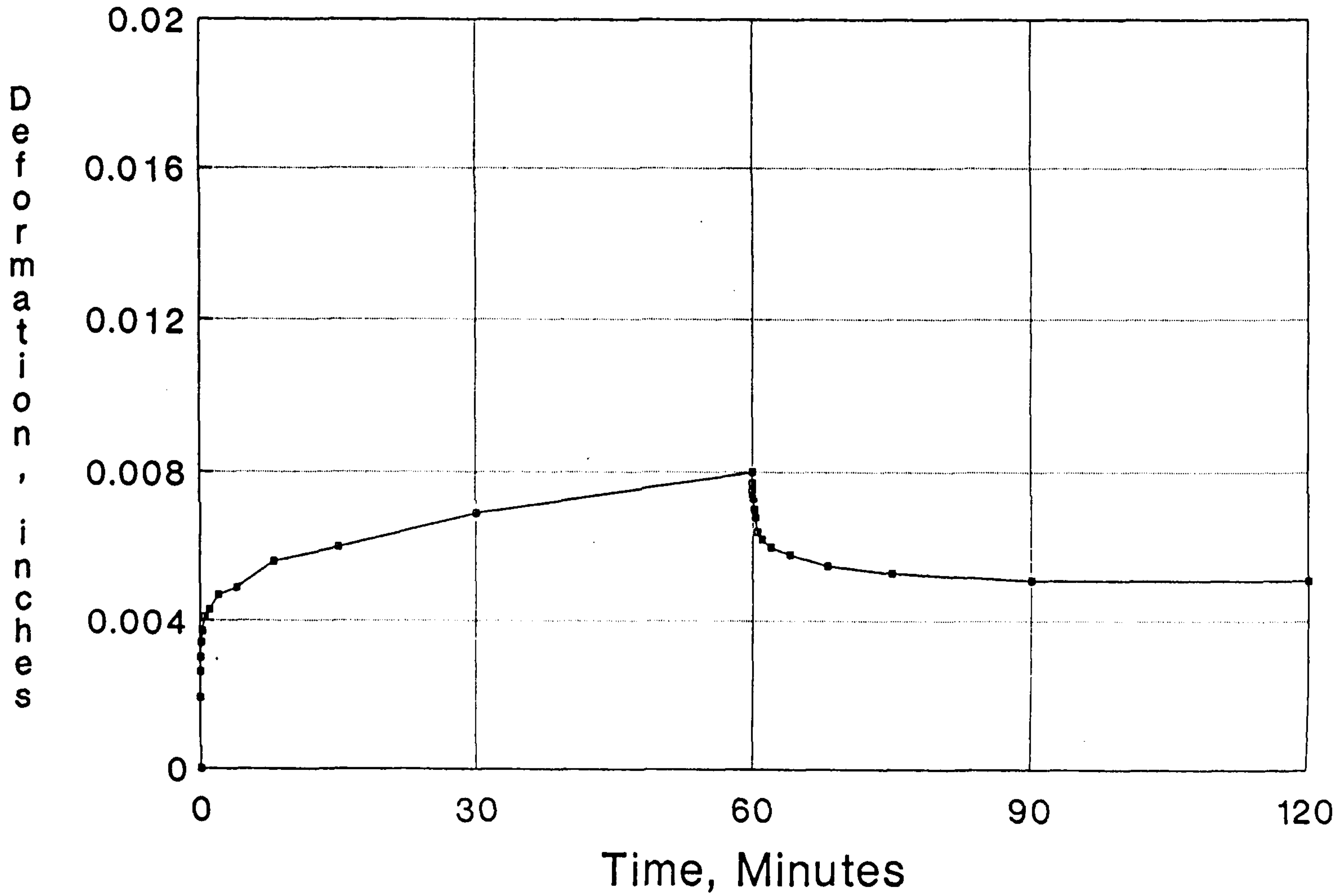


Figure 76. Creep-Rebound Curve for S-1C3 Sample at 104°F

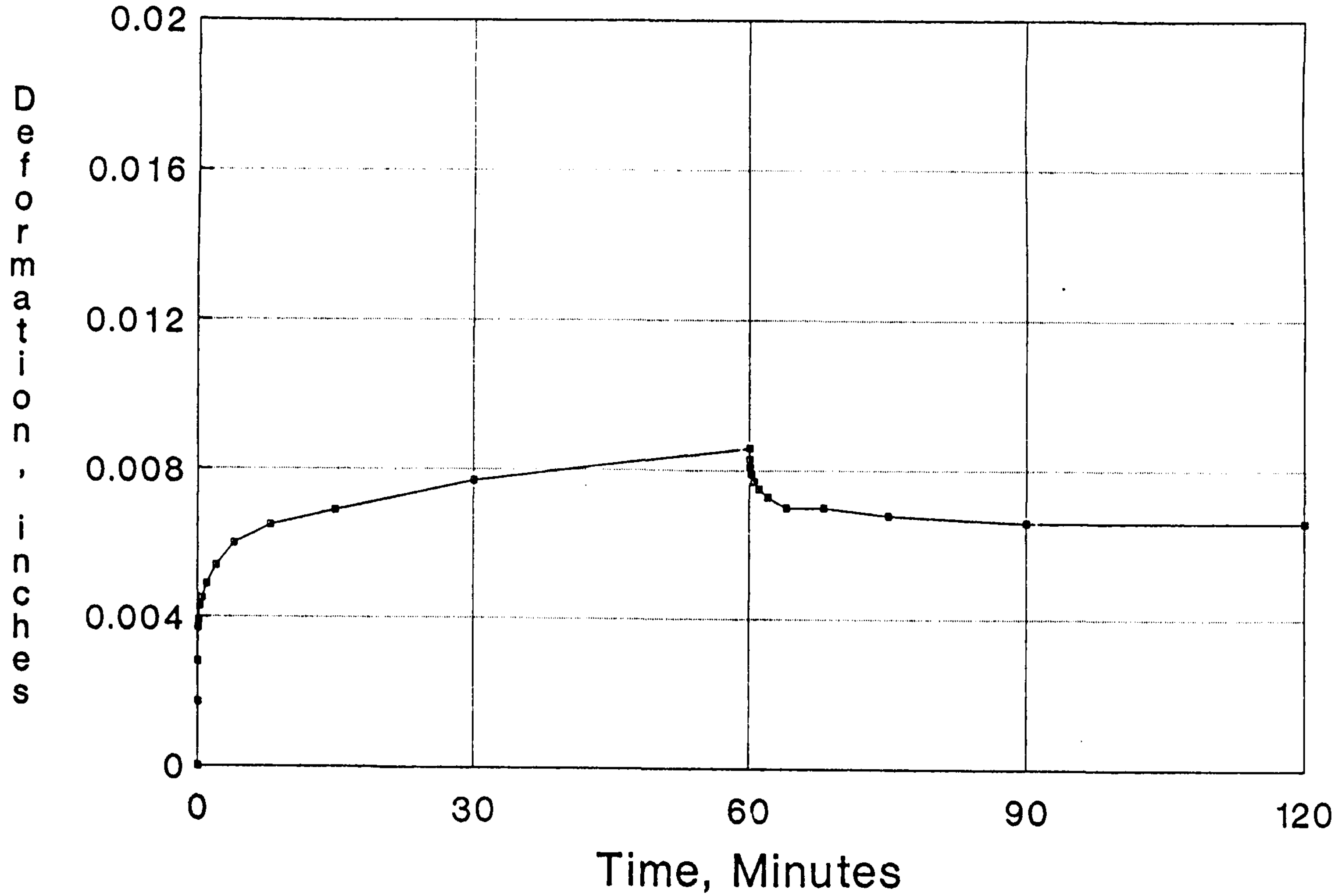


Figure 77. Creep-Rebound Curve for S-1C4 Sample at 104°F

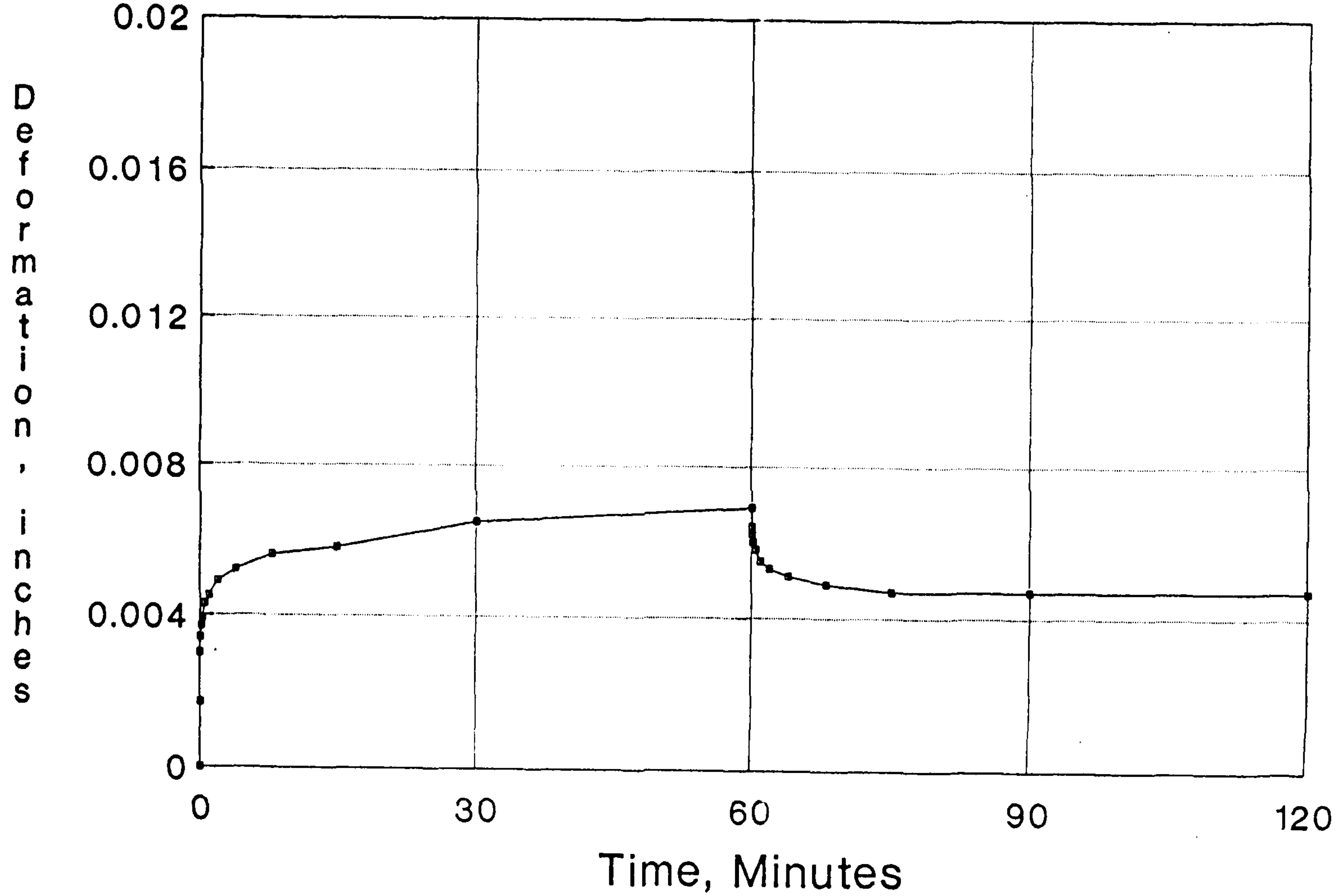


Figure 78. Creep-Rebound Curve for S-1C5 Sample at 104°F

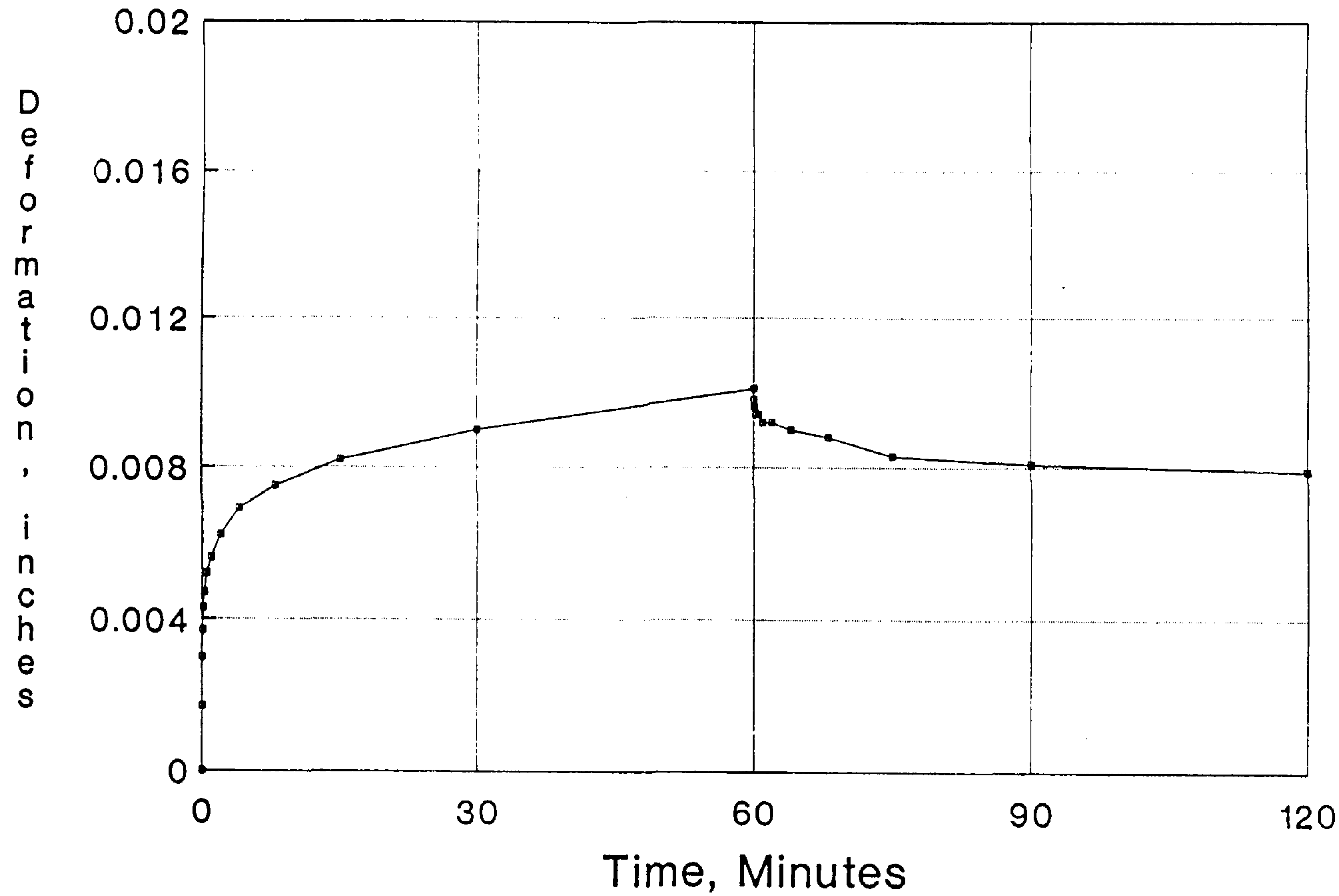


Figure 79. Creep-Rebound Curve for S-2C1 Sample at 77°F

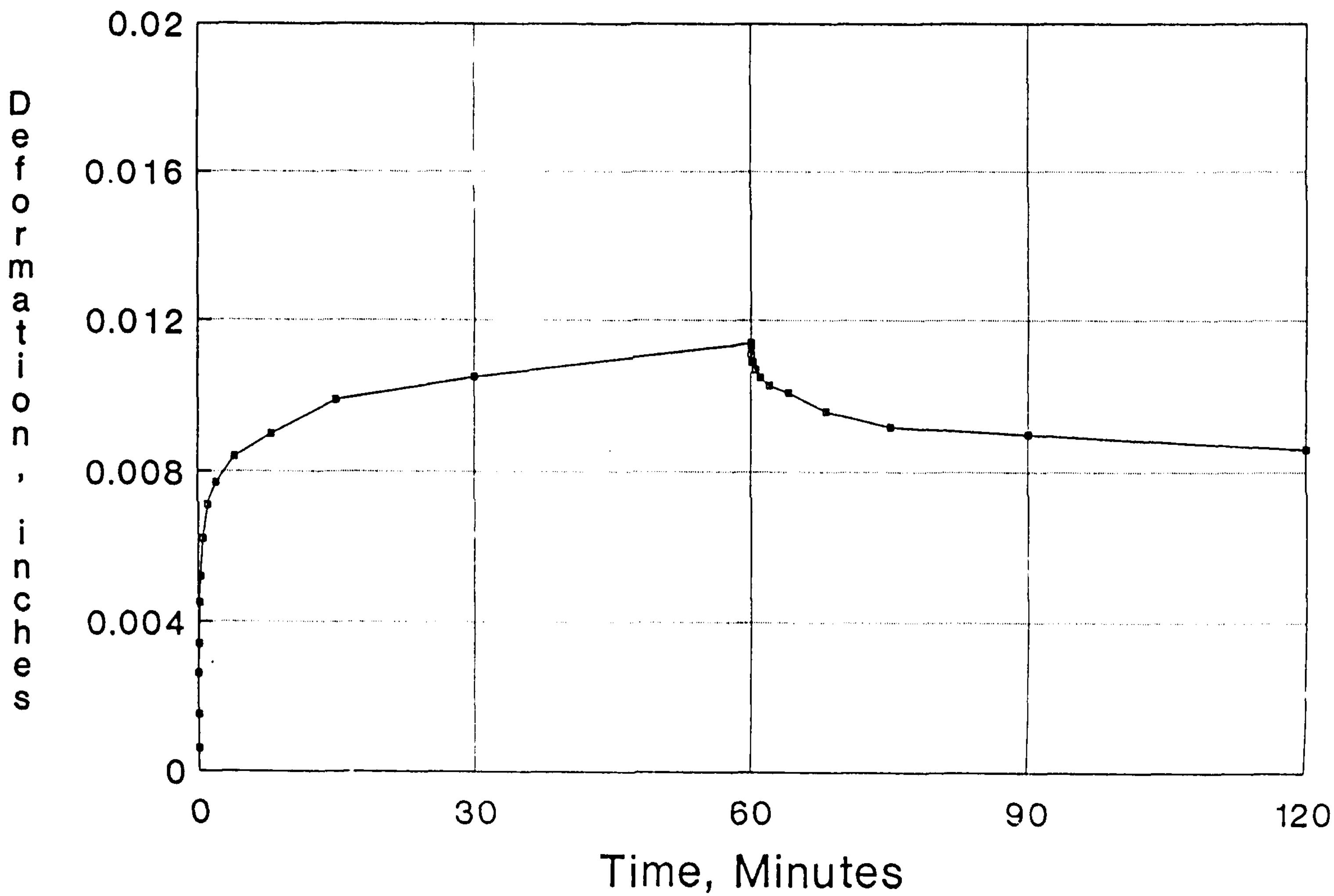


Figure 80. Creep-Rebound Curve for S-2C2 Sample at 77°F

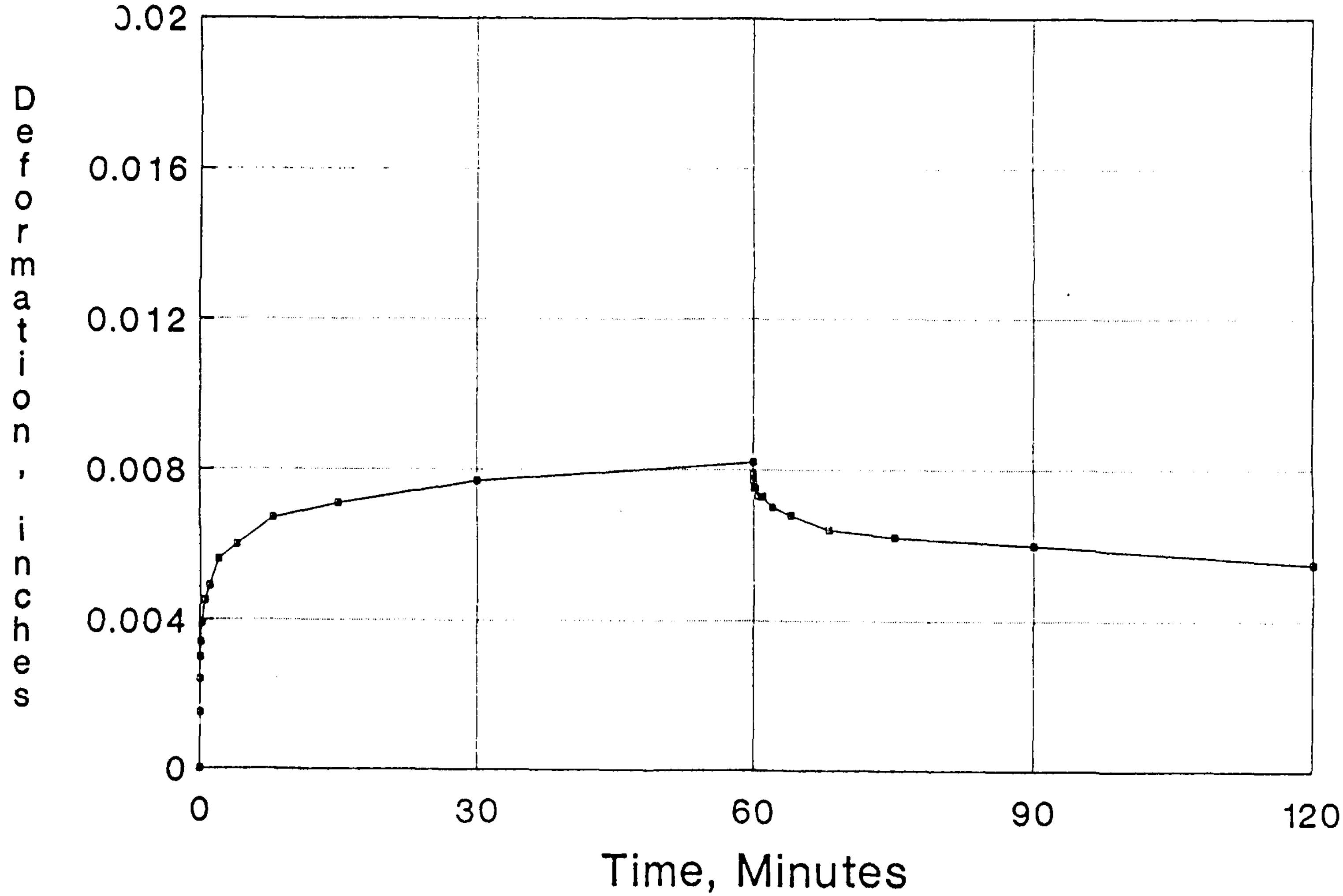


Figure 81. Creep-Rebound Curve for S-2C3 Sample at 77°F

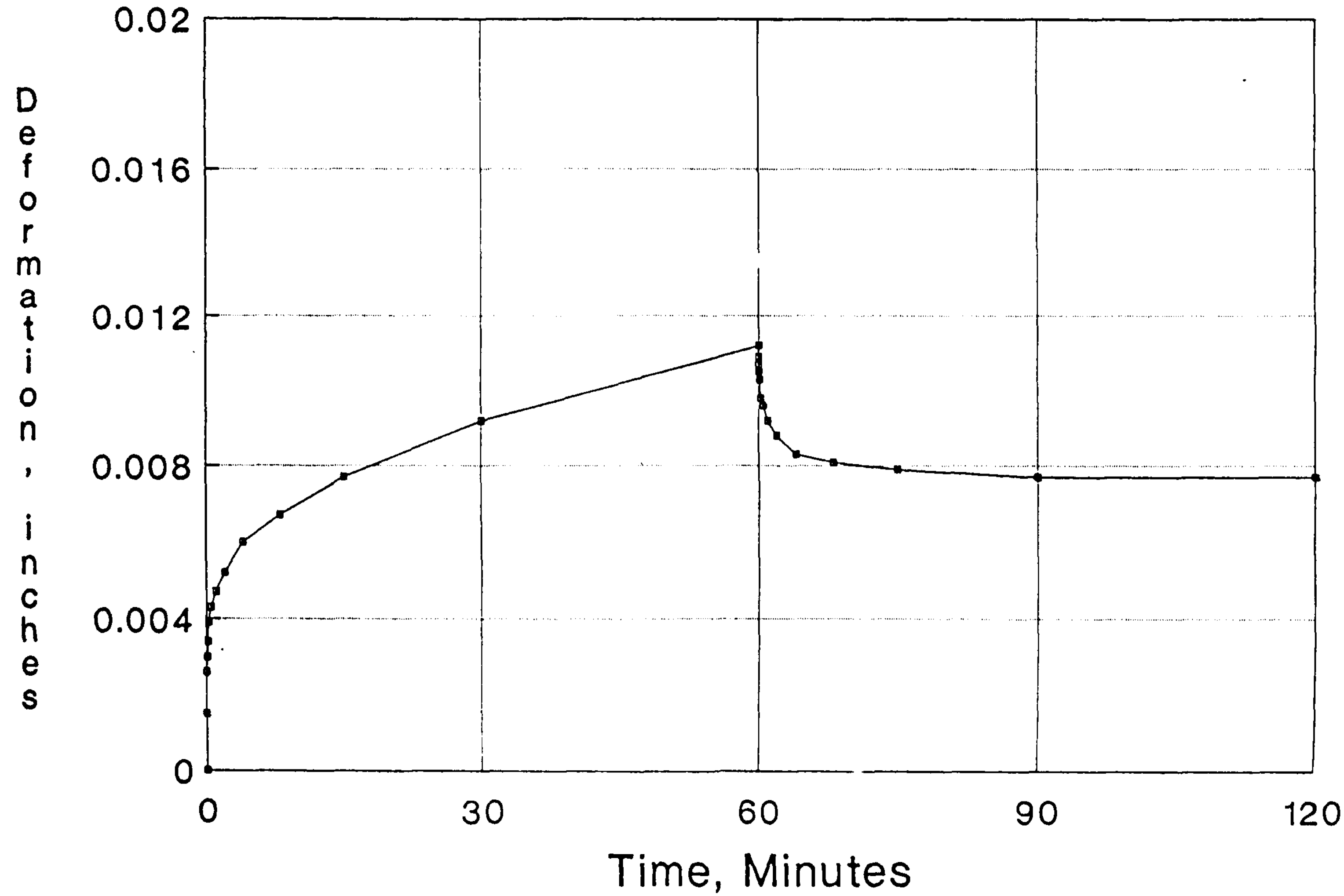


Figure 82. Creep-Rebound Curve for S-2C$ Sample at 104°F

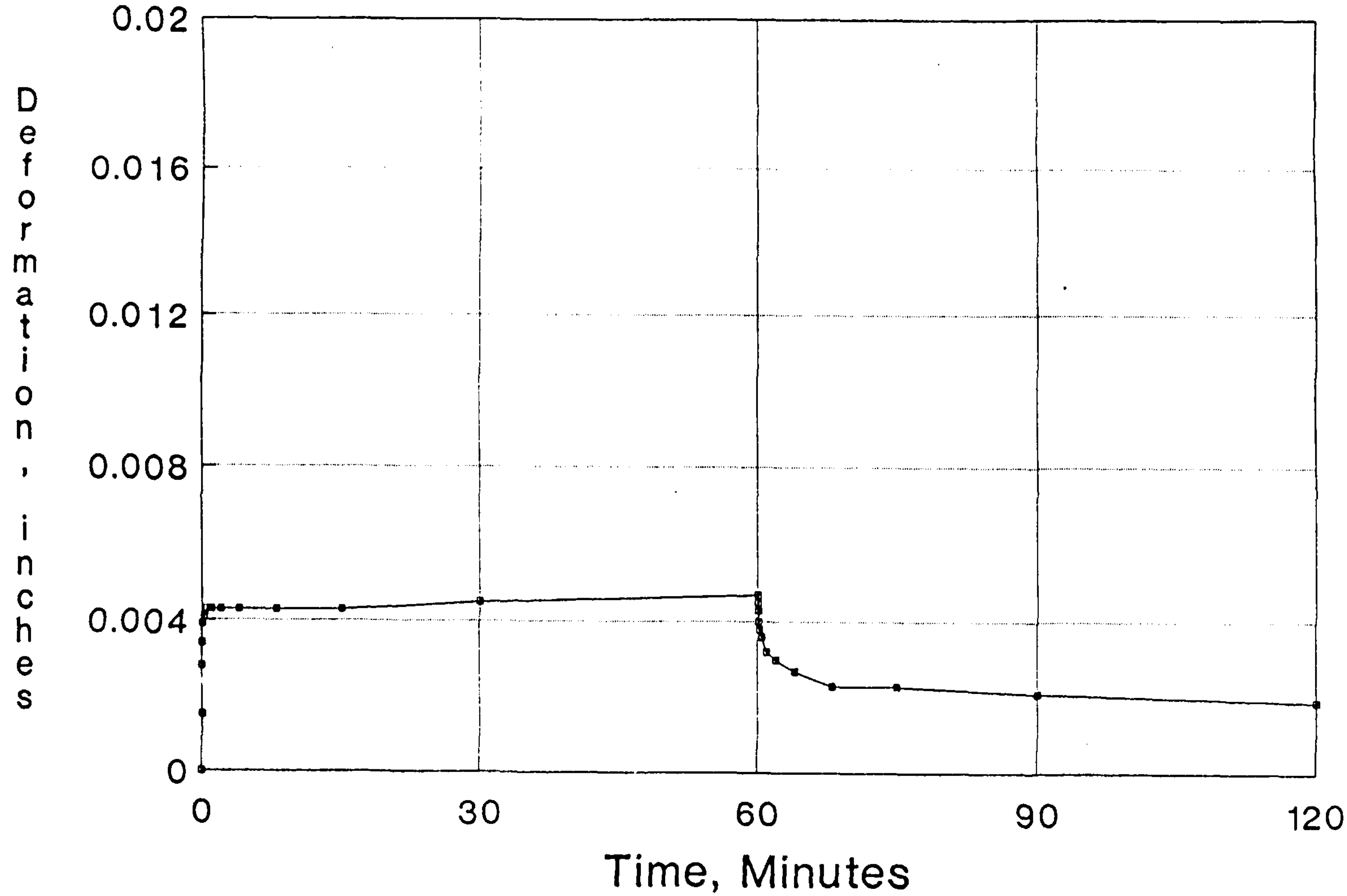


Figure 83. Creep-Rebound Curve for S-2C5 Sample at 104°F

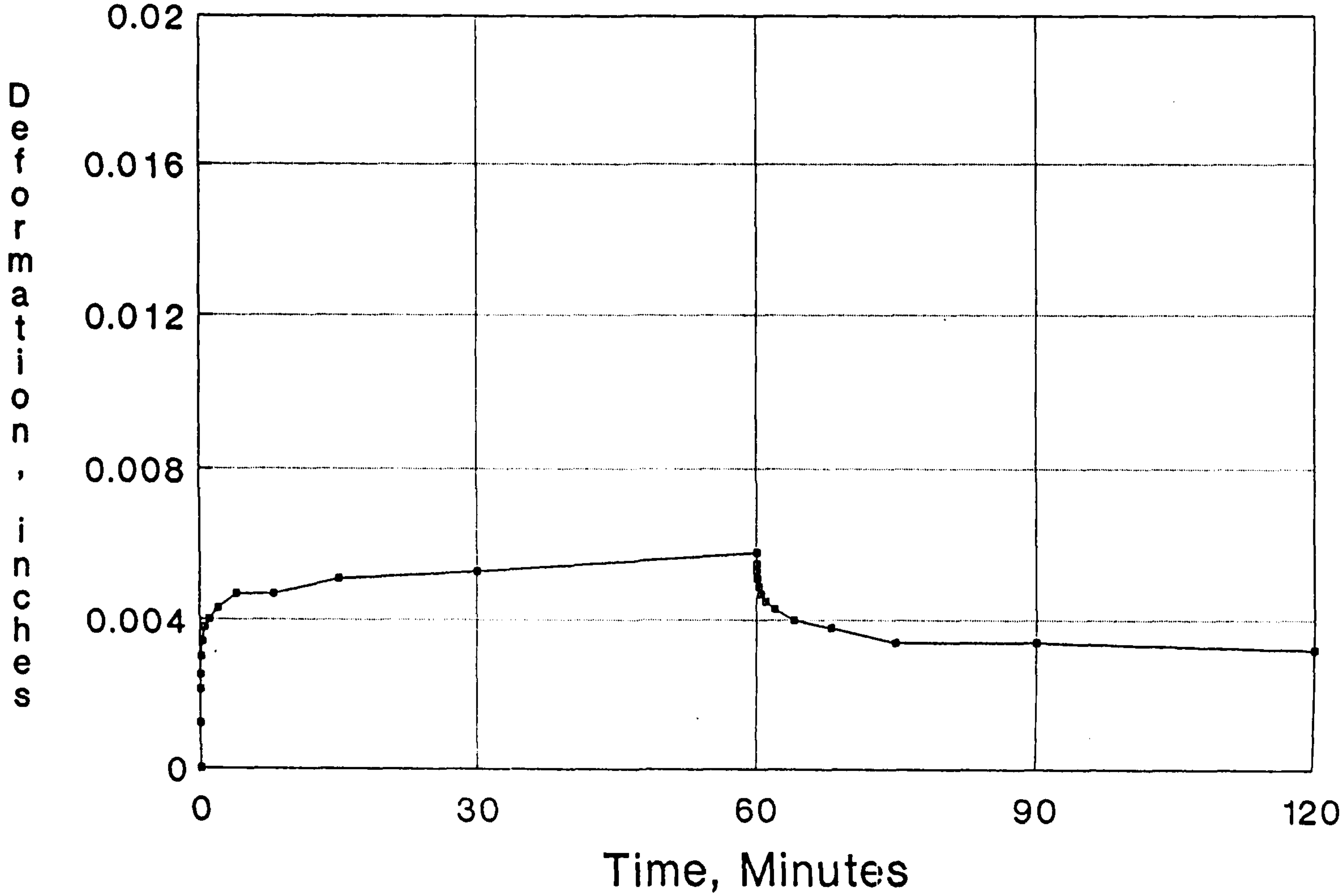


Figure 84. Creep-Rebound Curve for S-2C6 Sample at 104°F

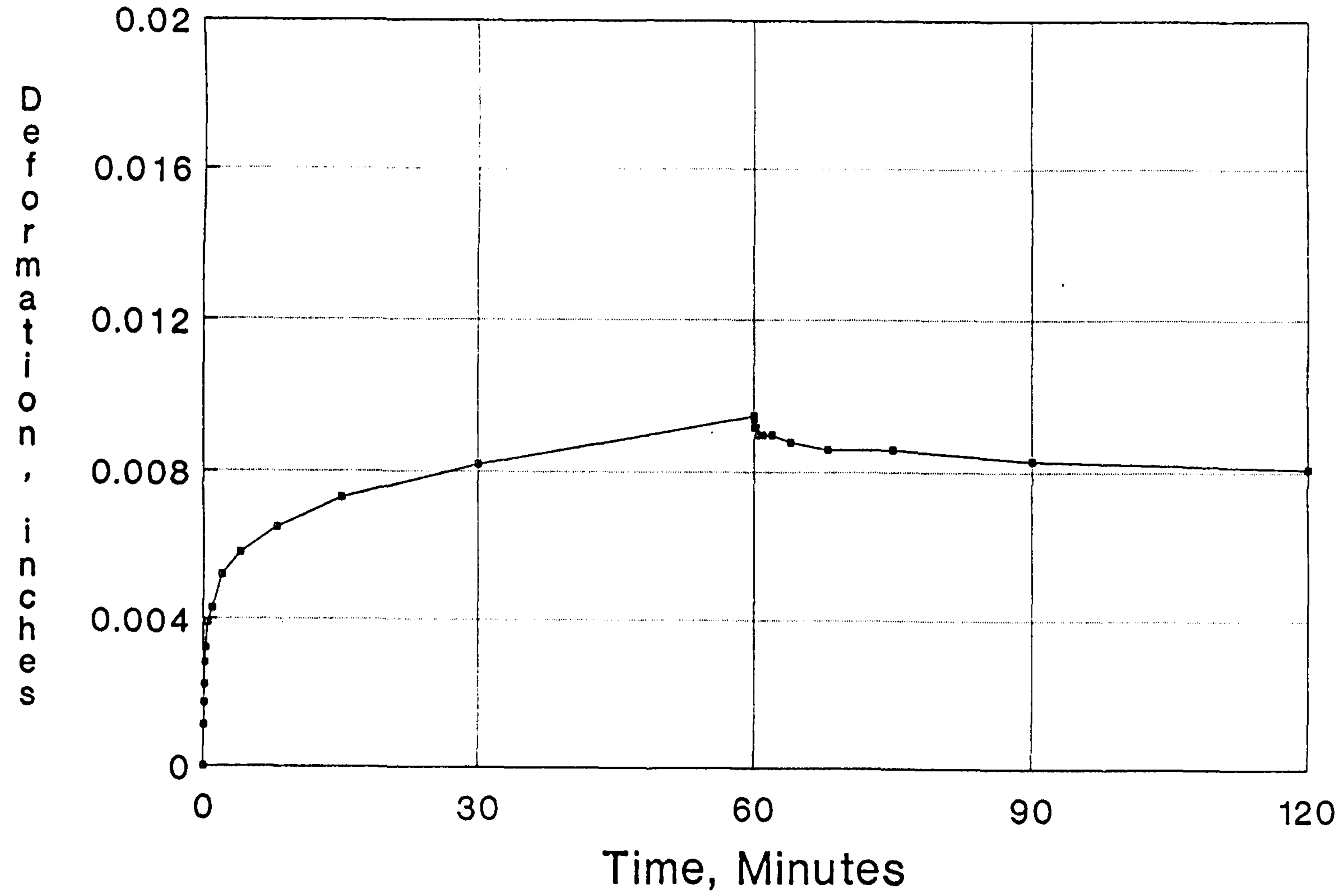


Figure 85. Creep-Rebound Curve for S-3C1 Sample at 77°F

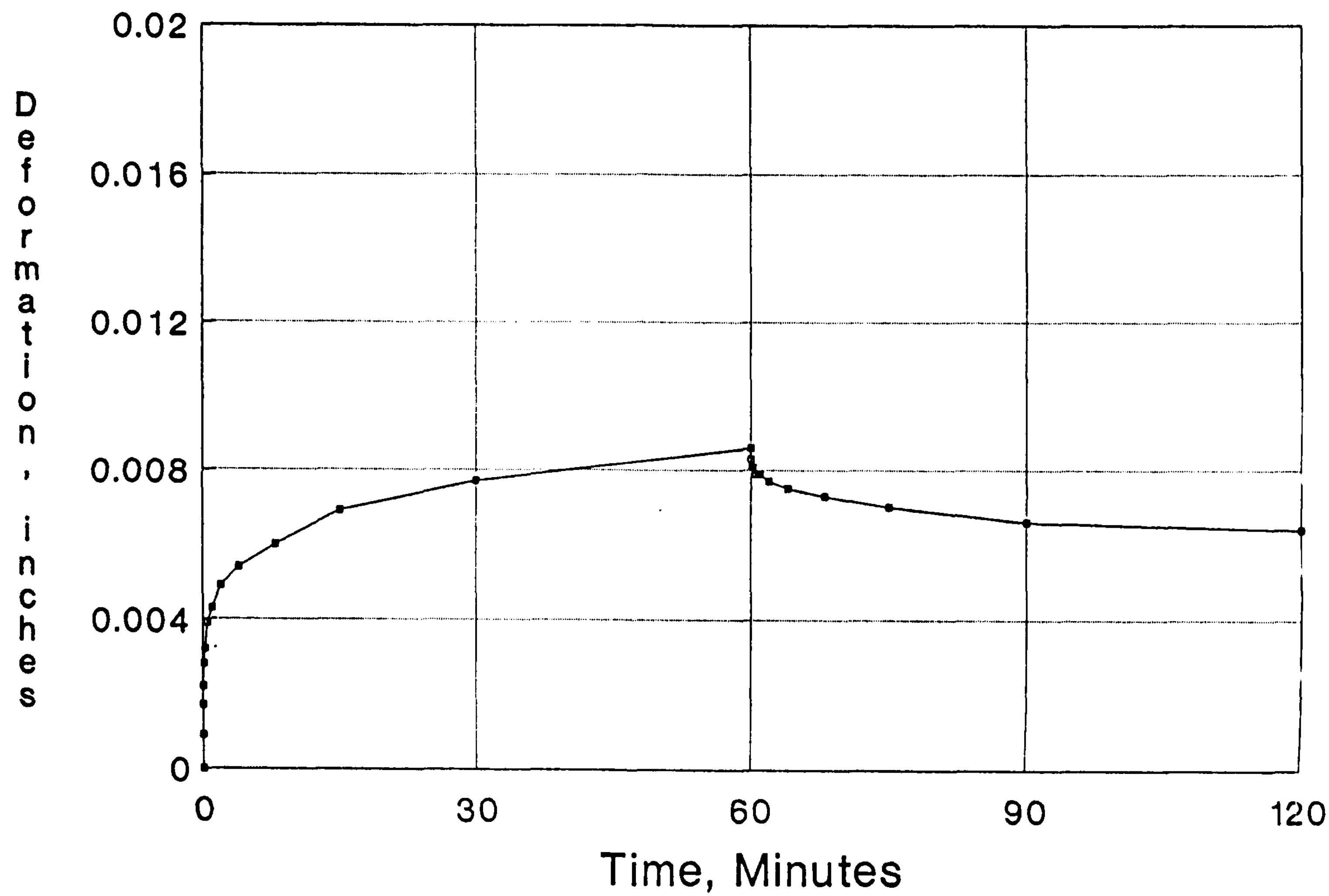


Figure 86. Creep-Rebound Curve for S-3C2 Sample at 77°F

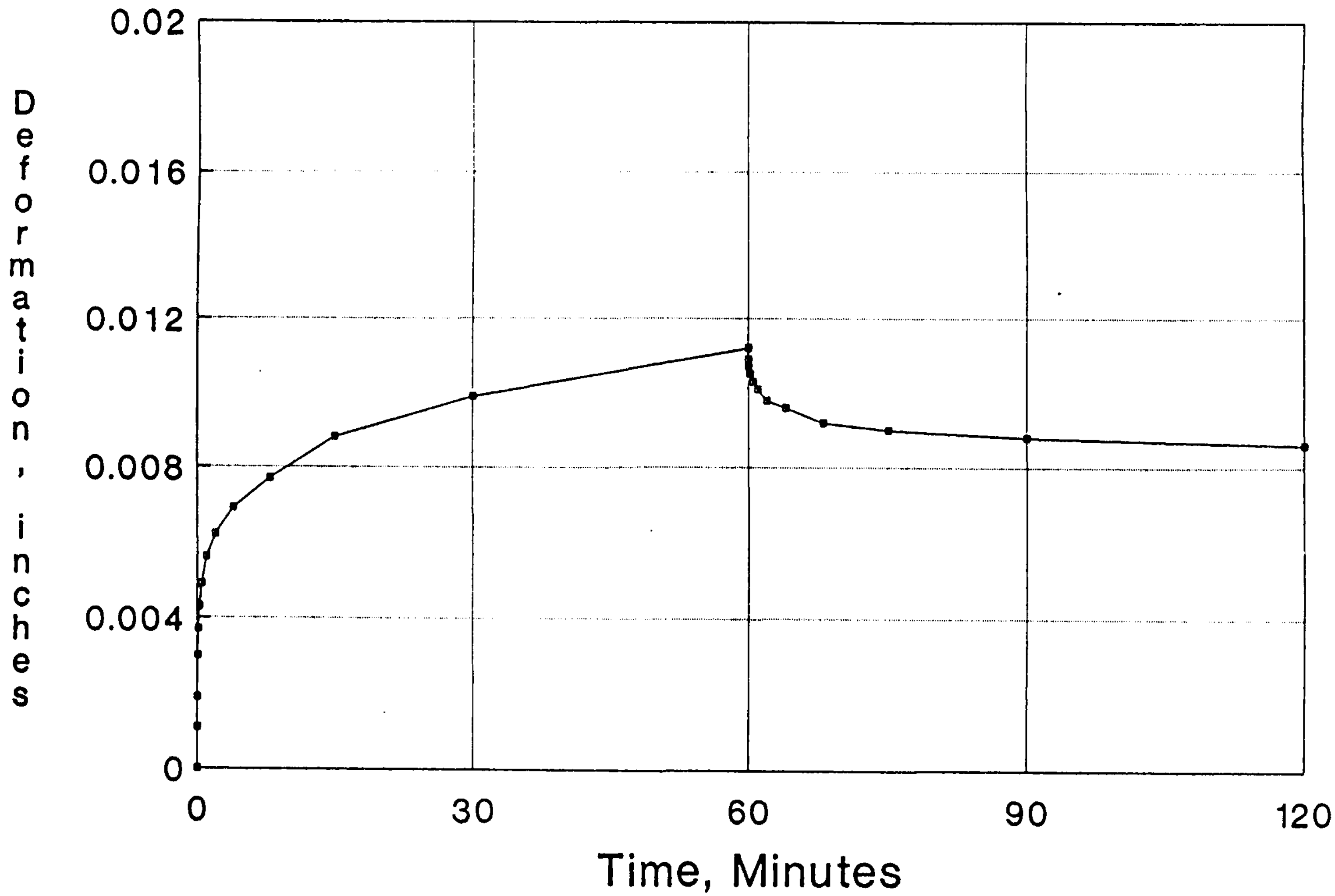


Figure 87. Creep-Rebound Curve for S-3C3 Sample at 77°F

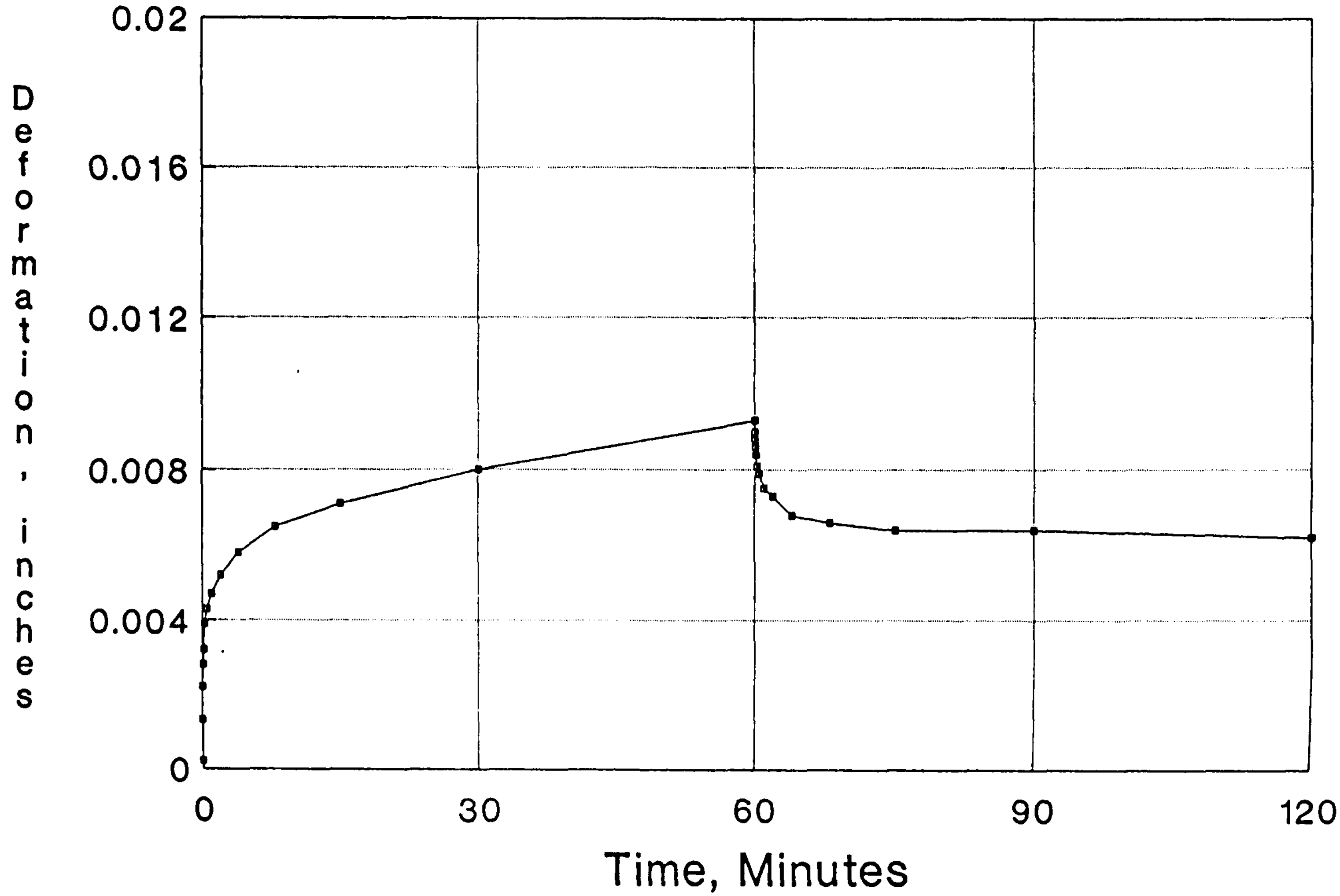


Figure 88. Creep-Rebound Curve for S-3C4 Sample at 104°F

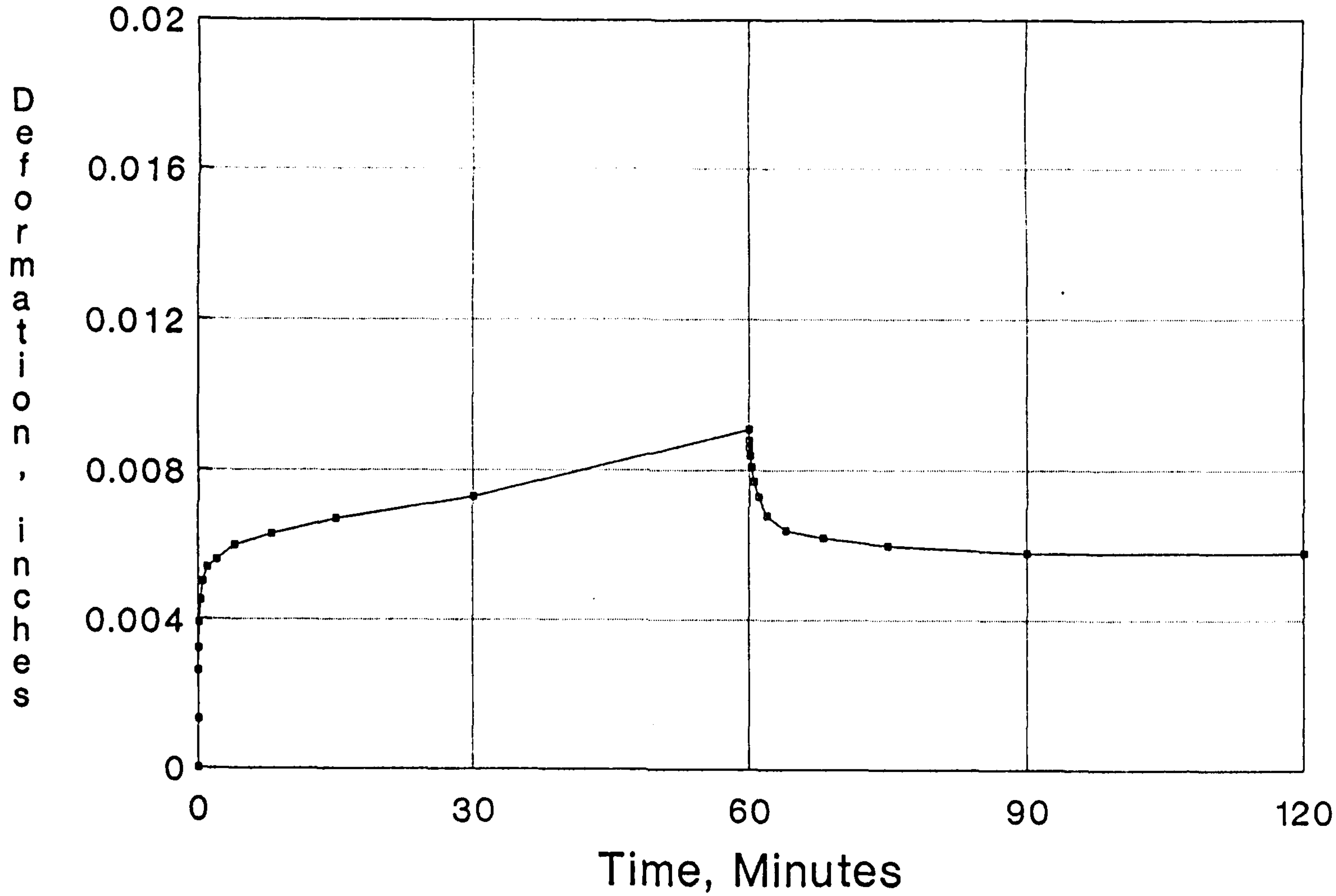


Figure 89. Creep-Rebound Curve for S-3C5 Sample at 104°F

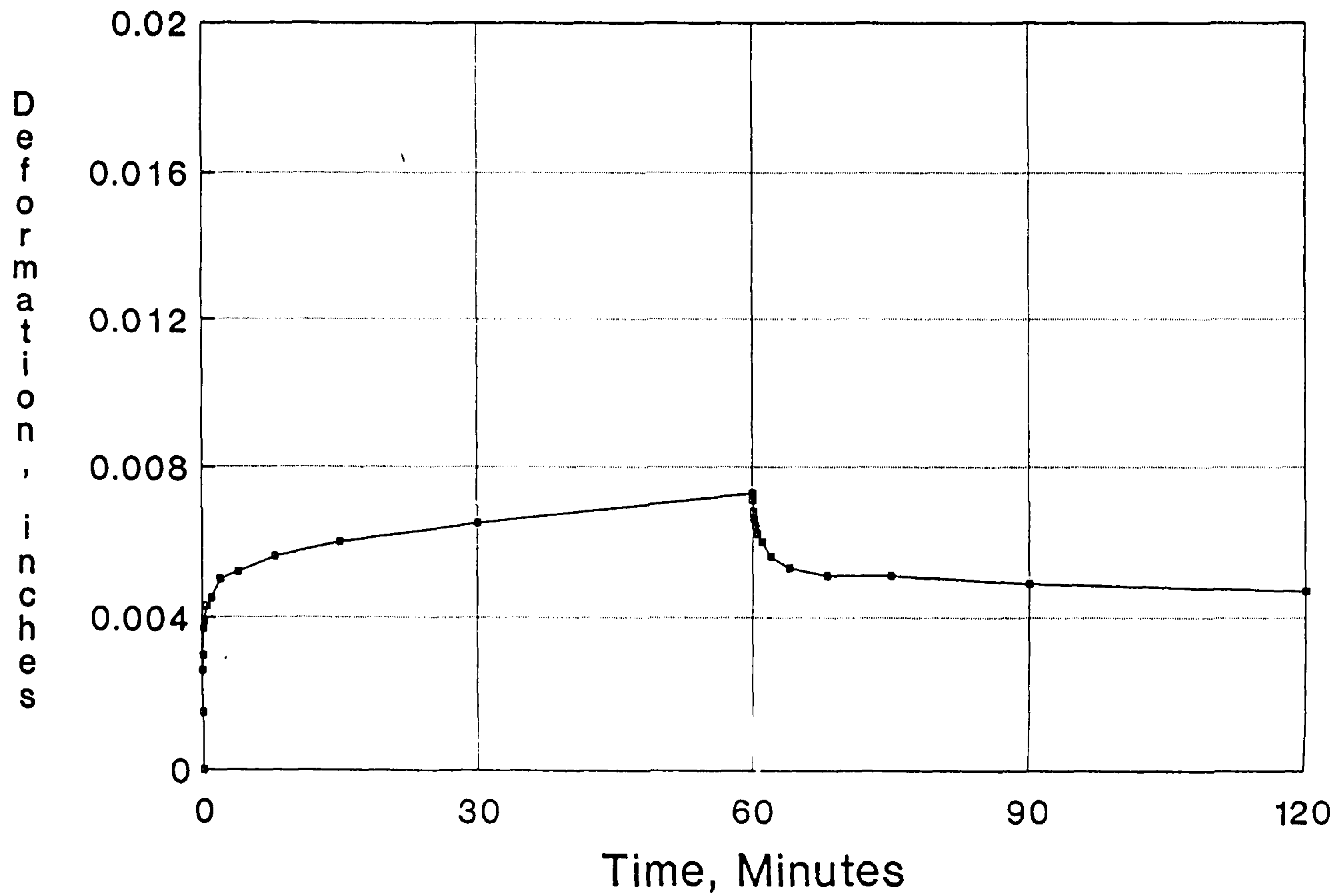


Figure 90. Creep-Rebound Curve for S-3C6 Sample at 104°F